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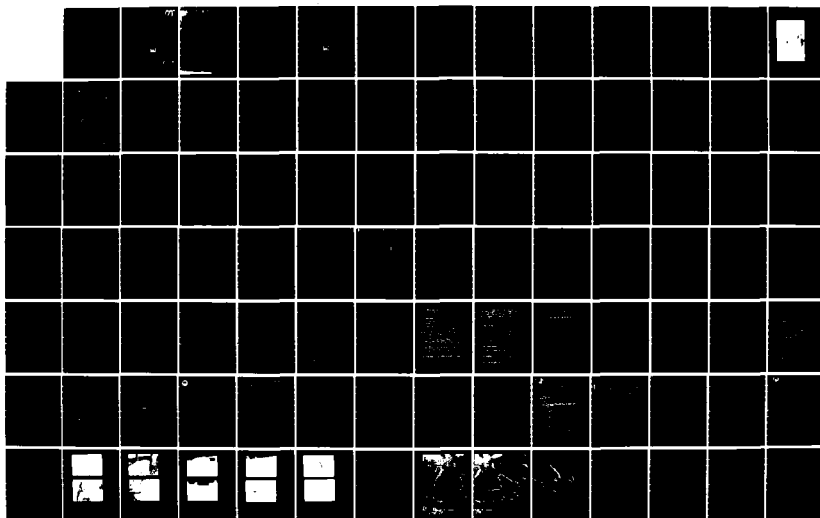
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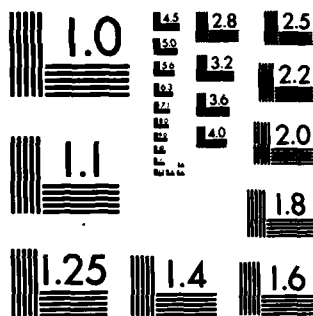
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AD-A143 307

CONNECTICUT RIVER BASIN  
NEW BRITAIN - SOUTHTON, CONNECTICUT

# SHUTTLE MEADOW RESERVOIR DAM CT 00162

## PHASE I INSPECTION REPORT NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY  
NEW ENGLAND DIVISION, CORPS OF ENGINEERS  
WALTHAM, MASS. 02154

MARCH 1979

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number) DAMS, INSPECTION, DAM SAFETY,  Conn. River Basin New Britian - Southington, Conn. Shuttle Meadow Reservoir Dam			
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The 560+ ft. long dam consists of an approx. 38 ft. high earth embankment and concrete coping with a concrete corewall along the axis of the dam. The total height from the corewall foundation to the top of the concrete coping is 51.3 ft. Based upon visual inspection at the site and past performance, the dam is judged to be in good condition. Based upon the size and hazard classification, the test flood will be equivalent to the PMF.			

CONNECTICUT RIVER BASIN  
NEW BRITAIN - SOUTHTON, CONNECTICUT

# SHUTTLE MEADOW RESERVOIR DAM CT 00162

## PHASE I INSPECTION REPORT NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY  
NEW ENGLAND DIVISION, CORPS OF ENGINEERS  
WALTHAM, MASS. 02154

MARCH 1979

## BRIEF ASSESSMENT

### PHASE I INSPECTION REPORT

#### NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	<u>SHUTTLE MEADOW RESERVOIR DAM</u>
Inventory Number:	<u>CT00162</u>
State Located:	<u>CONNECTICUT</u>
County Located:	<u>HARTFORD</u>
Town Located:	<u>NEW BRITAIN-SOUTHINGTON</u>
Stream:	<u>WILLOW BROOK</u>
Owner:	<u>NEW BRITAIN WATER COMPANY</u>
Date of Inspection:	<u>DECEMBER 6, 1978</u>
Inspection Team:	<u>PETER M. HEYEN</u>
	<u>CALVIN R. GOLDSMITH</u>
	<u>GONZALO CASTRO</u>
	<u>THOMAS O. KELLER</u>
	<u>HAROLD OLSEN</u>

The 560+ foot long dam consists of an approximately 38 foot high earth embankment and concrete coping with a concrete corewall along the axis of the dam. The total height from the corewall foundation to the top of the concrete coping is 51.3 feet. Upstream and downstream slopes are inclined to 2 horizontal to 1 vertical and 1.8 horizontal to 1 vertical, respectively. A concrete and brick gatehouse in the reservoir serves as an intake structure for a 30 inch cast iron supply main which runs under the dam to a downstream pump station. There is also said to be a 24 inch cast iron supply main under the dam, which was abandoned after the 30 inch main was put into service in 1893. The spillway is actually a concrete weir in a relatively flat, 7.5 foot deep, vertical-sided stone masonry channel. However, there are 4 foot high wooden stoplogs presently acting as a weir in the channel. There are two permanent diversions into the reservoir in the form of canals constructed in old streambeds. The east diversion passes near the right end of the dam and inlets to the reservoir upstream of the dam. The west diversion inlets at the left end of the dam.

Based upon the visual inspection at the site and past performance, the dam is judged to be in good condition. No evidence of instability of the embankment or appurtenant structures was observed. There are areas requiring attention.


Based upon the size (Intermediate) and hazard classification (High) of the dam in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to the Probable Maximum Flood (PMF), plus a portion of the runoff from each of the diversion areas equivalent to the capacity of each diversion canal. Peak inflow to the reservoir is 4300 cfs; peak outflow (Test Flood) is 1400 cfs with the dam overtopped 0.6 feet. The spillway capacity with the stoplogs in place, based upon our hydraulics computations, is 670 cfs, which is equivalent to 48 percent of the routed Test Flood Outflow.

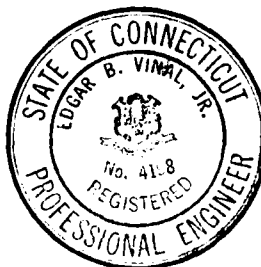
It is recommended that the owner remove the stoplogs immediately upon receipt of this report. It is also recommended that further studies be undertaken to perform a more refined hydraulic/hydrologic study to determine the best way to increase the ability of the spillway to pass a greater percentage of the Test Flood.

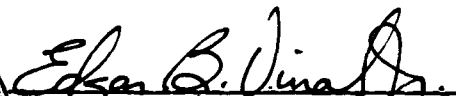
Further studies should be conducted by the owner pertaining to the seepage at the toe of the dam, the matter of possibly widening the top of the embankment, the hydraulic configuration of the inlet canals as they relate to the safety of the dam, and an evaluation of the outlet piping system, particularly the condition of the abandoned 24 inch cast iron supply main.

The recommendations and remedial measures above, and as described further in Section 7, should be instituted within 1 year of the owner's receipt of this report.



  
Peter M. Heyner, P.E.  
Project Manager Cahn  
Engineers, Inc.



  
Edgar B. Vinal, Jr., P.E.  
Senior Vice President  
Cahn Engineers, Inc.



This Phase I Inspection Report on Shuttle Meadow Reservoir has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

CHARLES G. TIERSCH, Chairman  
Chief, Foundation and Materials Branch  
Engineering Division

FRED J. RAVENS, Jr., Member  
Chief, Design Branch  
Engineering Division

SAUL C. COOPER, Member  
Chief, Water Control Branch  
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR  
Chief, Engineering Division

## PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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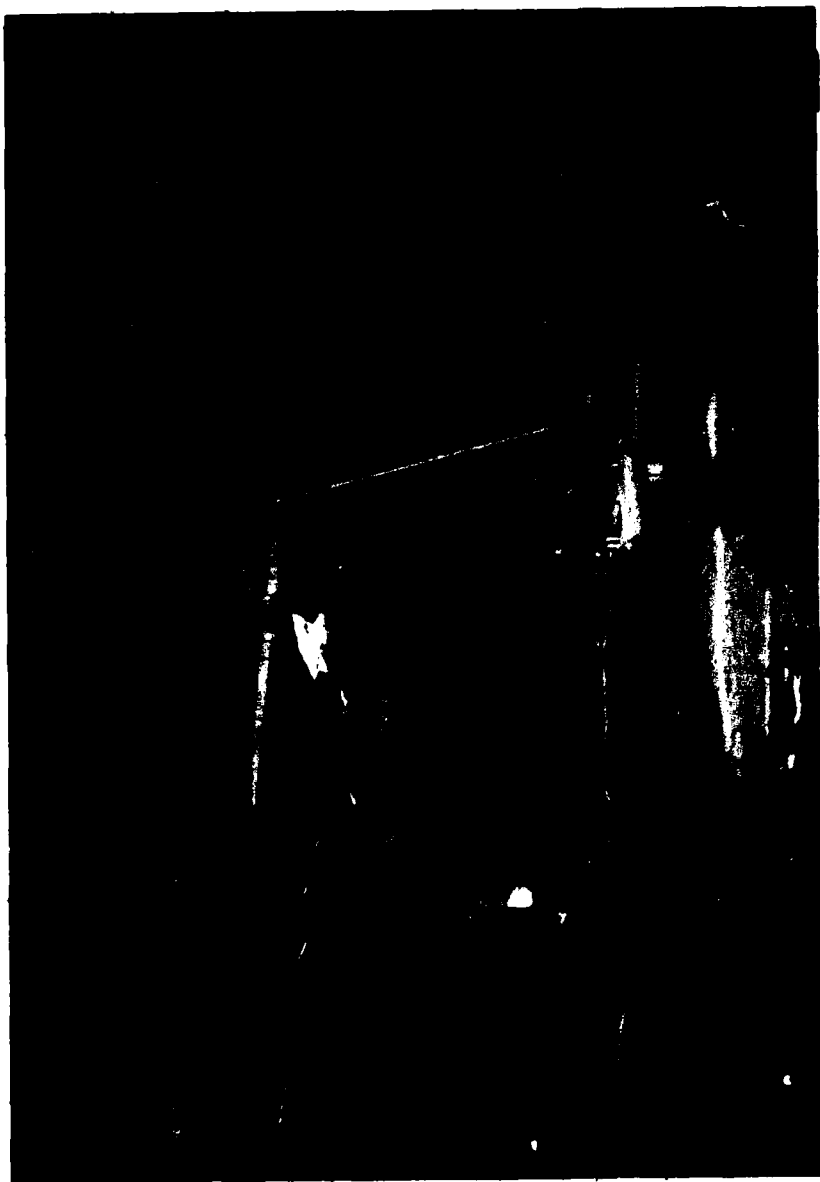
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OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED DAMS

Shuttle Meadow Res. Dam

Willow Brook

New Britain-  
Southington

CONNECTICUT

DATE Mar. 1979

CE # 27595 KA

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PHASE I INSPECTION REPORT  
SHUTTLE MEADOW RESERVOIR DAM

SECTION I  
PROJECT INFORMATION

1.1 GENERAL

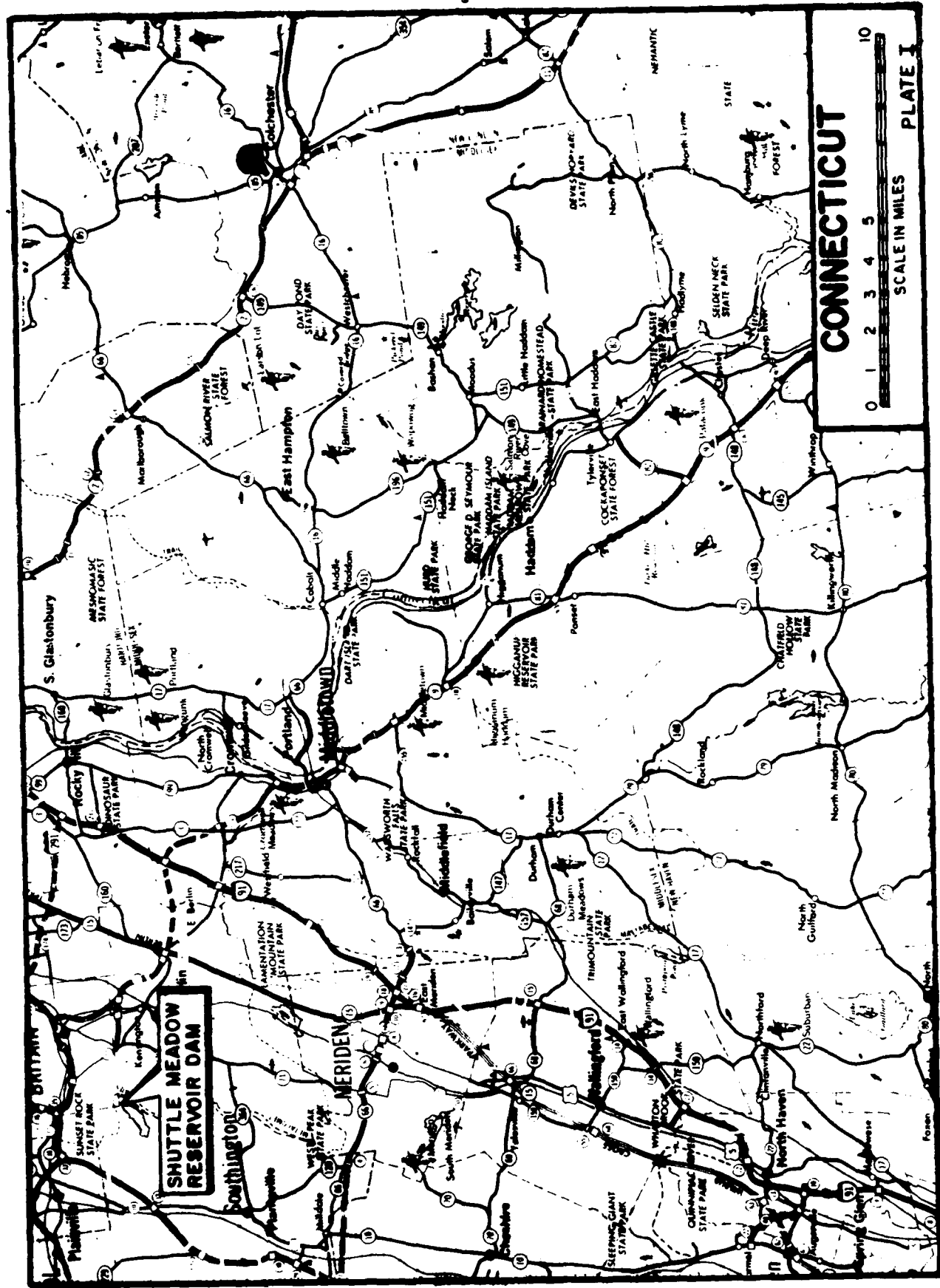
a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc, under a letter of November 28, 1978 from Max B. Scheider, Colonel, Corps of Engineers. Contract No. DACW 33-79-3-0014 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

- (1) Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interest.
- (2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.
- (3) To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

- (1) Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.
- (2) A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.





- (3) Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.
- (4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features on the dam which need corrective action and/or further study.

## 1.2 Description of Project

a. Location - The dam is located on Willow Brook in a rural area of the towns of New Britain and Southington, County of Hartford, State of Connecticut. The dam is shown on the New Britain U.S.G.S. Quadrangle Map having coordinates latitude N 41°38.7' and longitude W 72°49.2'.

b. Description of Dam and Appurtenances - The dam is an earth fill dam approximately 560+ feet long, the top of which is approximately 38 feet above the bed of Willow Brook. A concrete corewall 46.5 feet high is aligned along the axis of the dam and rises to approximately 4 feet below the 8 foot wide earth crest of the dam. A concrete coping wall 2 feet wide at the top, was built along the upstream edge of and 6 inches higher than the crest of the dam for the purpose of dissipating the force of wind-driven waves washing up against the upstream face and crest of the dam. The upstream slope of the dam, on an inclination of 2.0 horizontal to 1 vertical, is protected against erosion, from the coping wall down to the floor of the reservoir, by hand-placed riprap overlying a layer of broken stone. The downstream slope, at an inclination of 1.8 horizontal to 1 vertical, is covered with a heavy growth of grass. An earth berm, also grassed, runs along the toe of the slope. The bottom of the reservoir in the area of the gatehouse has been protected in a manner similar to the upstream slope to minimize turbidity of the water at the low level intake. The gatehouse, a brick and concrete structure in the reservoir itself, houses the intake to the 30 inch low level cast iron outlet pipe which feeds the pump station and the treatment plant, both downstream of the dam. The three intake ports to the gatehouse are controlled by heavy sluice gates located within the substructure of the gatehouse. A 24 inch cast iron pipe running from the previous gatehouse under the dam, was abandoned after the new gatehouse and outlet were constructed in 1893. There are valves and a system of interconnected pipes underground near the downstream toe of the dam which are accessible by means of a series of manholes and buried cylindrical brick structures.

There are two permanent diversions to the reservoir. One, at the left end of the dam, flows under the roadway which runs along the left side of the dam, and into the reservoir. There is a sidechannel off the diversion just upstream of the dam designed to divert water directly to Willow Brook without entering the reservoir. Another diversion at the right end of the dam has provisions for a cutoff and a gate to divert water into the spillway channel just before the concrete spillway sill. The spillway is actually a 7.5 foot high vertical sided masonry channel with 4 foot high stoplogs atop a series of concrete steps approximately at the mid point along the channel. At the end of the stone channel the water passes over a concrete sill to the hand-placed stone-lined channel which flows into Willow Brook.

c. Size Classification - INTERMEDIATE - The dam impounds approximately 5100 acre-feet of water at the top of dam elevation. According to the Recommended Guidelines, a dam with storage of between 1000 and 50,000 acre-feet of water is classified as being an intermediate size dam.

d. Hazard Classification - HIGH - Suburban developments of the City of New Britain, including the Lincoln School and Slade Junior High School, are located downstream on or near Willow Brook beginning approximately 1 mile from the dam.

e. Ownership - The New Britain Water Company  
The City of New Britain  
1000 Shuttlemeadow Avenue  
New Britain, Connecticut  
Mr. John McManus (203) 224-2491

f. Operator - Mr. John McManus (203) 224-2491

g. Purpose of Dam - Public Water supply.

h. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available.

A dam and gatehouse were built in 1857 to provide New Britain with a public water supply. In 1884, a canal was constructed at the east (right) end of the dam to divert water from the 0.7 square mile Panther Swamp watershed into the reservoir. In 1890, it was recognized that the reservoir was not going to provide an adequate water supply in the future for the rapidly growing City of New Britain, and construction of a new dam was authorized. The existing gatehouse and supply main were abandoned and the dam was completely removed. The present dam was constructed just downstream of the earlier dam with the present gatehouse and

supply main built as shown on the Shuttle Meadow Dam Plan sheet in Appendix Section B. Also constructed at this time was the west canal at the left end of the dam. The dam, canal and gatehouse were done by contract with the Troy Public Works Company of Troy, New York as designed and supervised by Percy M. Blake.

In 1912, the dam was raised 4 feet and the concrete coping wall was added. The concrete corewall was also raised 4 feet. At some time around the 1912 raising, the original gatehouse built in 1857 was removed.

The Connecticut Board of Civil Engineers, in a letter dated December 19, 1938, ordered the City of New Britain Board of Water Commissioners to among other things, increase the spillway capacity, widen the top of the dam from 9 to 20 feet, and remove all trees and saplings from the diversion canal embankments. The top of the dam was ordered widened to avoid future sloughing similar to that which occurred during the 1938 hurricane when spray from waves and wind washed over the dam onto the downstream slope. No record of these repairs or alterations was located.

i. Normal Operational Procedures - The 30 inch low level supply main from the gatehouse remains open supplying water to the pump station. In the infrequent event that the pump station is completely shut down, the supply main would be closed at the pump station leaving the intakes to the entrance of the main in the gatehouse open. The buried valves to the low level outlets, near the toe of the dam have not been utilized for as long as the water company engineer can remember. The four foot high stoplogs in the spillway channel usually remain in place, sometimes raised another 8 inches to provide increased storage for the reservoir.

### 1.3 Pertinent Data

a. Drainage Area - 2.94 square miles including the east and west canal diversions. Terrain is rolling and largely undeveloped.

b. Discharge at Damsite - Discharge from the reservoir is from a 30 inch cast iron supply main and from the spillway channel.

- |                                       |           |
|---------------------------------------|-----------|
| 1. Outlet works (conduit) size:       | 30 inches |
| 2. Invert El.:                        | N/A       |
| 3. Maximum known flood<br>at damsite: | N/A       |

4. Ungated spillway capacity @ top of dam:	670 cfs @ el. 380 <sub>+</sub>
5. Ungated spillway capacity @ Test Flood Elevation:	N/A
6. Gated spillway capacity at normal pool el.:	N/A
7. Gated spillway capacity @ test flood el.:	N/A
8. Total spillway capacity @ test flood el.:	N/A
9. Total project discharge @ test flood el.:	1400 cfs
c. <u>Elevation</u> (ft. above MSL)	
1. Streambed at centerline of dam:	342.5 (approx.)
2. Maximum tailwater:	N/A
3. West (left) diversion canal inlet:	372.5 <sub>+</sub>
4. East (right) diversion canal inlet:	372.5 <sub>+</sub>
5. Recreation pool:	N/A
6. Full flood control pool:	380.4
7. Spillway crest (gated):	374.4
8. Design surcharge (Original Design):	N/A
9. Top Dam:	380.4
10. Top of diversion canal embankments:	380
11. Test flood design surcharge:	N/A
d. <u>Reservoir</u>	
1. Length of maximum pool:	6000+ ft.
2. Length of normal pool:	6000 <sub>+</sub> ft.

3. Length of flood control pool:	N/A
e. <u>Storage</u>	
1. Recreation pool:	N/A
2. Flood control pool:	5100 ac.-ft.
3. Spillway crest pool (Top of Stoplogs):	4300 ac.-ft.
4. Test flood pool:	N/A
f. <u>Reservoir Surface</u>	
1. Top dam:	230 ac.
2. Test flood pool:	N/A
3. Flood-control pool:	N/A
4. Recreation pool:	N/A
5. Spillway crest:	183 ac. (See Appendix Section D-11)
g. <u>Dam</u>	
1. Type:	Earthfill with concrete corewall
2. Length:	560 ft.
3. Height:	Structural=51.3 ft.; 38 ft. above original meadow level.
4. Top Width:	10 ft.
5. Side Slopes:	2H to 1V upstream 1.8H to 1V down- stream
6. Zoning:	N/A
7. Impervious Core:	Hand mixed and placed concrete corewall
8. Cutoff:	N/A
9. Grout curtain:	N/A

10. Other:	Concrete coping wall at upstream crest of dam
h. <u>Diversion and Regulating Tunnel</u>	
1. Type:	2 diversion canals
2. Length:	N/A
3. Closure:	N/A
4. Access:	To right and left of dam
5. Regulating Facilities:	Gates to channel by-passing reservoir
i. <u>Spillway</u>	
1. Type:	Stone masonry channel with concrete sill
2. Length of weir:	18 ft. (across channel)
3. Crest el. (stoplogs):	374.4
4. Gates:	4 foot high wooden stoplogs
5. U/S Channel:	Shallow 20 ft. wide approach channel
6. D/S Channel:	Shallow discharge channel
7. General:	N/A
j. <u>Regulating Outlets</u>	
1. Invert:	N/A
2. Size:	30 inch diameter
3. Description:	Cast iron pipe. Supply main from reservoir.

4. Control Mechanism:

Gate to intake  
structure. Valve at  
downstream toe of  
dam embankment.

5. Other:

Abandoned 24 inch  
cast iron supply  
main.

## SECTION 2: ENGINEERING DATA

### 2.1 Design

a. Available Data - The available data consists of drawings by the City of New Britain, and correspondence by Percy M. Blake, the Connecticut Board of Civil Engineers, Edward W. Bush and Clarence M. Blair both members of the Connecticut Board of Civil Engineers, William E. Tyler of the City of New Britain, Arthur L. Shaw of Metcalf and Eddy, J. J. Curry, B.L. Bigwood of the Water Resources Division of U.S. Geologic Survey, B. H. Palmer of Chandler and Palmer, George W. Wood of the City of New Britain, and an article which appeared in the New Britain section of the Hartford Courant in December 1978.

b. Design Features - The data and correspondence indicate the design features stated previously in this report.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the original construction or the 1912 raising, other than information on watershed areas feeding the reservoir.

### 2.2 Construction

a. Available Data - As built drawings were not available for the dam, nor were any actual construction records.

b. Construction Considerations - No information was available.

### 2.3 Operations

Lake level readings are taken daily. The dam spillway capacity has not been exceeded to our knowledge.

### 2.4 Evaluation

a. Availability - Existing data was provided by the State of Connecticut and the owner. The owner also made the dam available for our visual inspection.

b. Adequacy - The limited amount of detailed engineering data available was generally inadequate to perform an in-depth assessment of the dam, therefore, the final assessment of this dam must be based primarily on visual inspection, performance history, and hydraulic computations of spillway capacity based upon approximate hydrologic judgements.



c. Validity - A comparison of record data and visual observations reveals no observable significant discrepancies in the record data.

### SECTION 3: VISUAL INSPECTION

#### 3.1 Findings

a. General - The general condition of the dam is good. Inspection did reveal some areas requiring attention. The reservoir water level was at elevation 361.2, 19.7 feet below the top of the coping wall, at the time of our inspection.

b. Dam:

Crest - The crest is grass covered and has a concrete coping wall along the upstream edge as shown in Photo 1. No misalignment of the crest was observed as judged by the coping wall. Small depressions (probably tire tracks) on the crest due to trespassing vehicles were observed. Spalling of the concrete coping wall has occurred as can be seen in Photo 2.

Upstream Slope - The upstream slope is covered with hand placed riprap and is generally in very good condition as shown in Photo 1. There are two small areas near the left abutment and near the top of the slope where riprap is missing.

Downstream Slope - A view of the downstream slope in Photo 7 shows it to be covered with a heavy growth of grass, as is the berm along the toe of the slope also seen in Photo 7. A gentle hump was observed in the berm near the center of the dam. Several small areas of minor erosion a few inches deep were observed beneath the grass on the slope. Minor trespassing on the downstream slope was observed.

Two wet zones were observed near the two ends of the dam immediately downstream of the berm as shown on the dam plan sheet in Appendix Section B. The zone near the left abutment was approximately 35 feet by 25 feet in area. The zone near the right abutment was approximately 45 feet by 25 feet in area and can be seen in Photos 7 and 8.

A standpipe observation well with an inside diameter of 2.5 inches was found in the wet zone at the right end of the dam (Photo 9). Clear water was observed flowing from the pipe at a rate of 0.04 gallons per minute as measured by timing the rise of water in the pipe after removing a few inches of water. A sounding of the pipe indicated it had a depth of 21 feet from the ground surface, which is the same depth given in a letter dated August 15, 1963. In that letter, a description of a small amount of clear water coming from the well seems to indicate no significant siltation of the well has occurred since 1963.

Stones were observed at the intersection of the downstream toe of the berm and the left abutment as shown in Photo 10. It is not known whether the observed stones are part of a toe drain. Also near the intersection of the downstream toe and the left abutment there is a vertical 24 inch diameter steel pipe sticking up about 1 foot above the ground surface and filled with soil to within a few feet from the top. The purpose of this pipe is not known.

There is a possible rock outcrop or boulder downstream of the downstream toe of the embankment. Approximately 5 square feet of the rock was visible.

Spillway - The walls of the spillway approach channel and discharge channel are of stone masonry construction and are in good condition (Photos 5 and 6). There are a few stones and some mortar dislodged from the face of the walls. Several large pieces of cut stone are resting on the channel floor. They are designed possibly to dissipate the effects of water striking the channel walls. The stone blocks are extraneous to the spillway, i.e., they were not dislodged from or originally part of the spillway or channel, and hence do not present a problem, other than impeding flow in the spillway channel. The floor of the channels are of stone with some grass growing through. The wooden stoplogs in the spillway channel appear very sound and in good condition.

c. Appurtenant Structures - There are several cracks in a concrete wall supporting the roadway passing over the left diversion canal inlet to the reservoir. The most severe cracking is to the right of the canal inlet as shown in Photos 3 and 4. The cracks have offset varying amounts, the largest amount being about 1.2 inches. The roadway pavement adjacent to the cracked area is in good condition.

The diversion canals at each end of the dam appear to be in good condition. In the case of the left canal, provisions have been made for the installation of a stoplog gate in the canal to divert water into Willow Brook downstream of the dam, thus bypassing the reservoir; however, no stoplog gate was in evidence and the gate to the sidechannel diversion from the main diversion appeared undersized. For the right canal, provisions for a stoplog gate are also provided, though no gate is in evidence. The sidechannel from this canal flows to the spillway channel downstream of the spillway stoplogs by means of a low barrier and gate which only constrict the bottom 2 feet of the sidechannel.

The buried valves to the low level outlet pipes near the downstream toe of the dam were not operated for inspection. Mr. John McManus reported that the valves are not used now and have not been used in the 10 years he has been with the New Britain Water Company.

d. Reservoir Area - The reservoir area is bordered on the west and partially on the southeast by roadways. The area surrounding the reservoir is wooded, except at the very south end, and largely undeveloped. The two diversions feeding the reservoir are also in wooded, largely undeveloped areas.

e. Downstream Channel - The spillway discharge channel and the left diversion canal sidechannel join together downstream of the center of the dam and pass between two 8 to 10 foot high, narrowly separated stone walls which support a narrow bridge. From there, the channel becomes the streambed of Willow Brook.

### 3.2 Evaluation

Based upon the visual inspection, it was possible to assess the dam as being generally in good condition. The following features which could influence the future condition and/or stability of the dam were identified.

1. Wet areas at the downstream toe of the dam should be monitored periodically for increased seepage. The water flow from the observation well should also be monitored.
2. Cracking of the concrete wall supporting the roadway should be repaired. The displacement should be monitored periodically for any worsening of the condition.
3. The small amount of stones and mortar missing or deteriorated in the spillway channel sidewalls should be replaced or repaired.
4. Spalling of the coping wall will increase if not repaired.
5. The small depressions on the crest of the dam are not a problem, but should be monitored for worsening of the condition.
6. The valves to the low level outlet pipes are located on the downstream side of the dam which means that even in the closed position, the pipe running through the dam is full of water under pressure.

## SECTION 4: OPERATIONAL PROCEDURES

### 4.1 Regulating Procedures

The intake sluice gates to the gatehouse and 30 inch supply main are always open, feeding to the pump station. Should the flow from the supply main be stopped, which happens infrequently, the main would be closed at the pump station rather than the gatehouse. The buried gate valves at the downstream toe of the embankment are not used, and it is not known by the owner which valves open which pipelines. The condition of the abandoned 24 inch supply main is not known.

### 4.2 Maintenance of Dam

Maintenance of the dam is on an as-needed basis and is carried out by the caretaker. To our knowledge, no formal maintenance procedures are in existence. Periodic maintenance includes replacing the stoplogs approximately every 5 years.

### 4.3 Maintenance of Operating Facilities

There is no maintenance done for the operating facilities of the dam. It is our impression that the only operating facilities maintained are at the pump station and the treatment plant.

### 4.4 Description of any Formal Warning System in Effect

The dam is watched closely during storms by the caretaker who lives on the grounds. Any blockage of the canals or spillway, or other emergency maintenance would be done by the owner immediately. In the event of an emergency, the New Britain Police Department would be notified.

### 4.5 Evaluation

The operation and maintenance procedures require improvement, including the present practice of closing off the low level supply main from the gatehouse at a point downstream of the dam rather than at the gatehouse on the upstream side of the dam. The condition of the 24 inch abandoned supply main should be determined, particularly as to whether or not the pipeline is sealed off upstream of the dam or downstream, or possibly closed only by means of the buried gate valves at the downstream toe of the dam.

A formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a more sophisticated formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Further operation and maintenance recommendations are presented in Section 7.

## SECTION 5: HYDRAULIC/HYDROLOGIC

### 5.1 Evaluation of Features

a. General - The reservoir may be described as a high surcharge storage-low spillage type project. Along with the two canal diversions, there are numerous other pipe diversions which allow Shuttle Meadow Reservoir to be used to supplement the storage of some of the other reservoirs in the water supply system which have greater drainage areas but much less storage. Although not often utilized, these diversions include lines into Shuttle Meadow Reservoir from Whigville Reservoir, Whitesbridge Pump Station, Wolcott Reservoir, and the Patton Brook pump station.

Both canal diversions into the reservoir have gates which can be opened to allow some of the inflow to be diverted directly to Willow Brook downstream of the dam prior to entering the reservoir. In the event of heavy flows, however, the gates are not adequate to handle the maximum canal capacities, and there are no provisions to stop the canal's flow into the reservoir, or to keep the reservoir from flooding the canals. (Refer to Section 3.1c) The actual capacity of the canals is limited to the flow contained to the top of the low hillside banks, because the canals are intercepting canals for most of their length and run parallel to the contours of their intercepted hillside watersheds. Therefore, the inflow from the canals is considerably lower than the PMF of their intercepted watershed, and varies with the water level in the reservoir and the corresponding backwater and available freeboard before the canals are overtopped.

b. Design Data - The project was designed to provide a water supply for the rapidly expanding City of New Britain of 1891. There were no computations found for the original construction in 1891 or for the 1912 raising of the dam.

c. Experience Data - No information on serious problem situations arising at the dam were found, and it does not appear the dam has been overtopped. During the 1938 hurricane, the high water level combined with the wind and waves spraying water over the top of the dam and causing erosion of the sod and surface slumping of the downstream slope.

d. Visual Observations - Although provisions for the installation of stoplogs in both diversion canals have been made, there were no gates or stoplogs in evidence to regulate the flow of water into the reservoir via the canals. There are four foot high stoplogs in the spillway canal and stone blocks on the canal bottom, both of which serve to reduce the spillway capacity.

e. Test Flood Analysis - The Test Flood for this high hazard, intermediate size dam is equivalent to the Probable Maximum Flood (PMF) (for the reservoir drainage area) of 2700 cfs plus a portion of the 2 diversion area runoffs equivalent to the inflow capacity of the 2 diversion canals, which amount to 800+ cfs for each canal. Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges", dated March, 1978, peak inflow to the reservoir is 4300 cfs (Appendix D-10); peak outflow (Test Flood) is 1400 cfs with the dam overtopped 0.6 feet (Appendix D-12). Based upon our hydraulics computations, the spillway capacity is 670 cfs, which is approximately 48% of the routed Test Flood Outflow with the water level at the top of dam, elevation 380.4. Parallel computations assuming only the inflow from the reservoir drainage area without the diversions are included in Appendix Section D.

f. Dam Failure Analysis - Utilizing the April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 64,000 cubic feet per second. A breach of the dam would result in a 15.4 foot high wave approximately one mile downstream at the beginning of suburban residential developments which include the Lincoln School and Slade Junior High School.



## SECTION 6: STRUCTURAL STABILITY

### 6.1 Evaluation of Structural Stability

a. Visual Observations - The visual inspections did not disclose any indication of stability problems. There was cracking of the concrete roadway bridge over the left diversion canal, as described in Section 3. There were no indications of recent cracking of adjacent portions of the bridge or of the roadway, which was in good condition. Spalling was also observed on the coping wall, as described in Section 3.

b. Design and Construction Data - The design and construction data is not sufficient to permit an in-depth analysis of the stability of the dam.

c. Operating Records - Records indicate that in 1938, the hurricane caused waves to spray over the top of the dam resulting in numerous areas of erosion of the sod and surface slumps on the downstream slope.

d. Post Construction Changes - The dam was raised four feet and the concrete coping wall was built in 1912. The raising of the dam narrowed the top of the dam from the original 20 feet to the present day 10 feet. The narrowed crest is what allowed spray from the 1938 hurricane to reach the downstream slope and cause its erosion.

An August 1963 inspection of the dam indicated the existence of the two wet areas that were observed during this inspection. The right wet zone was reported to be 75 feet square and the left zone was reported to be about 20 feet square. In 1963, the caretaker reportedly recalled that the wet areas had been in existence for some years before 1963. Thus, it appears that the wet areas observed in the present inspection have been in existence since before 1963.

The above described seepage condition does not appear to have had a measurable effect on the stability of the dam.

It is not known whether the berm on the downstream slope was part of the original construction or a post-construction change.

e. Seismic Stability - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

## SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

### 7.1 Dam Assessment

a. Condition - Based upon the visual inspection of the site and past performance, the dam appears to be in good condition. No evidence of structural instability was observed in the dam or its appurtenances. There are areas requiring attention including the spillway capacity, the diversion canals and their capability of being adequately diverted away from the dam, and the general hydraulic configuration of the dam relative to the diversion canals. Other recommendations and remedial measures are presented in Sections 7.2 and 7.3, respectively.

Based upon the "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, peak inflow to the reservoir is 4300 cubic feet per second; peak outflow (Test Flood) is 1400 cubic feet per second with the dam overtopped 0.6 feet. Based upon our hydraulics computations, the spillway capacity is 670 cubic feet per second, which is equivalent to approximately 48 percent of the Test Flood.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based solely on visual inspection, the past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within one year of the owner's receipt of this report.

d. Need for Additional Investigation - There is a need for additional investigation of the dam as described in Section 7.2.

### 7.2 Recommendations

1. Based upon the Phase I computations in Appendix D, the dam spillway capacity will be exceeded by the Test Flood. More sophisticated flood routing should be undertaken by hydrologists/hydraulics engineer to refine the Test Flood figures. A study should be undertaken and recommendations made by a registered professional engineer to increase the spillway capacity based upon the refined Test Flood figures. The study should include an examination of the effect of the removal of the 4 foot high stoplogs on the spillway capacity.

A registered professional engineer qualified in dam design and inspection should perform the following investigations:

2. Inspection of the dam when the reservoir level is high. A measurement of the water flow from the standpipe observation well at the right downstream toe of the embankment should be made to determine if the volume of flow from the well is directly related to the water level in the reservoir. An evaluation of the significance of any flow increase noted should then be undertaken.
3. Address the matter of widening the top of the dam embankment to some minimum width as was recommended previously by the Connecticut Board of Civil Engineers in a letter dated December 19, 1938. Recommendations as to the necessity for widening, and for the amount of any widening of the crest should be made by the engineer.
4. A registered engineer qualified in dam design and hydraulics should be retained to evaluate the hydraulic configuration of the diversion canals as they relate to the safety of the dam. It appears that the hydraulic characteristics are deficient in two areas:

First, an adequate means of diverting the incoming flow of the diversion canals to the brook downstream of the dam must be devised to cut down inflow to the reservoir during large storms. The present side channels and gates are inadequate and/or too poorly aligned to handle the flow during a Test Flood. Any means of closing off the diversion canals must be easily accessible and be able to be put into effect rapidly in the event of an emergency.

Second, as the canals are on a very flat slope, a method of closing off the canals at or very near the outlets to Shuttle Meadow Reservoir should be devised, to prevent high water in the reservoir from moving back up the canal and down the side slope should failure of one of the canal embankments occur.

5. Undertake an investigation to determine what condition the piping system, particularly the abandoned 24 inch supply main, is in. If the 24 inch main was not sealed and was shut off only by the valves at the downstream toe of the dam, then

the pipe would be under a constant head of water through the dam and recommendations should be made to seal the pipe at the upstream toe of the dam. The proper method of operation should be determined for the series of interconnected pipes and valves buried near the downstream toe of the dam. The condition of the 30 inch supply main should also be ascertained.

### 7.3 Remedial Measures

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time indicated in Section 7.1.c, and continued on a regular basis where applicable.

1. The stoplogs in the spillway channel should be removed by the owner immediately upon receipt of this report.
2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference. The program of operations should be modified such that when the flow from the supply main is to be shut off in any but the most temporary instances, the upstream gatehouse inlets should be utilized rather than using only the valves at the pump station. Also, a more formalized program of monitoring of the dam during storms should be instituted.
3. The cracks in the concrete bridge wall over the left diversion canal inlet should be repaired. The spalling of the concrete coping wall and the small amount of stones and mortar missing from the spillway channel sidewalls should also be repaired.
4. The cutting of grass on the crest and downstream slope should be continued as part of the routine dam maintenance.
5. Riprap missing on the upstream slope should be replaced.
6. Sluice gates to the supply main intake chamber of the gatehouse should be maintained on a regular basis to render them easily operable.

7. A formal program of inspection by a registered professional engineer should be instituted on an annual basis. The inspections should be technical in nature and should include the operation of any functioning low level outlets.

#### 7.4 Alternatives

This study has identified no practical alternatives to the above recommendations and remedial measures.

**APPENDIX**  
**SECTION A: VISUAL OBSERVATIONS**

# VISUAL INSPECTION CHECK LIST

## PARTY ORGANIZATION

PROJECT SHUTTLEMEADOW RES. DAM

DATE: 12/6/78

TIME: 2:00 PM

WEATHER: SUNNY 40°

W.S. ELEV. 361.2 U.S. \_\_\_\_\_ DN.S

### PARTY:

### INITIALS:

### DISCIPLINE:

1. <u>PETER M. HEYNE</u>	<u>PMH</u>	<u>CAHN ENGINEERS, INC.</u>
2. <u>CALVIN R. GOLDSMITH</u>	<u>CRG</u>	<u>CAHN ENGINEERS, INC.</u>
3. <u>GONZALO CASTRO</u>	<u>GC</u>	<u>GEOTECHNICAL ENGINEERS, INC.</u>
4. <u>THOMAS KELLER</u>	<u>TK</u>	<u>GEOTECHNICAL ENGINEERS, INC.</u>
5. <u>HAROLD OLSEN</u>	<u>HO</u>	<u>NEW BRITAIN WATER CO.</u>
6. _____	_____	_____

### PROJECT FEATURE

### INSPECTED BY

### REMARKS

1. <u>EARTH DAM EMBANKMENT</u>	<u>PMH, CRG, GC, TK</u>
2. <u>DIVERSION CHANNELS INTO RESERVOIR</u>	<u>PMH, CRG, GC, TK</u>
3. <u>OUTLET CONTROL TOWER</u>	<u>PMH, CRG, GC, TK</u>
4. <u>LOW LEVEL PIPES/CONDUITS</u>	<u>PMH, CRG, GC, TK</u>
5. <u>SPILLWAY AND CHANNELS</u>	<u>PMH, CRG, GC, TK</u>
6. <u>CONCRETE BRIDGES</u>	<u>PMH, CRG</u>
7. _____	_____
8. _____	_____
9. _____	_____
10. _____	_____
11. _____	_____
12. _____	_____

# PERIODIC INSPECTION CHECK LIST

Page A-2

PROJECT SHUTTLEMEADOW RES. DAM

DATE 12/6/78

PROJECT FEATURE EARTH DAM EMBANKMENT BY PMH, CRG, GK, TK

AREA EVALUATED	CONDITION
<u>DAM EMBANKMENT</u>	
Crest Elevation	380.9
Current Pool Elevation	~ 20' BELOW CONCRETE CAP ON CREST OF DAM
Maximum Impoundment to Date	NA
Surface Cracks	NONE OBSERVED
Pavement Condition	ONLY ALONG LEFT ABUTMENT GOOD CONDITION
Movement or Settlement of Crest	SOME SLIGHT SETTLEMENT BEHIND CONCRETE CAP DUE TO VEHICULAR TRAFFIC.
Lateral Movement	NONE OBSERVED
Vertical Alignment	APPEARS GOOD
Horizontal Alignment	APPEARS GOOD
Condition at Abutment and at Concrete Structures	GOOD - NO APPARENT EROSION
Indications of Movement of Structural Items on Slopes	NONE OBSERVED
Trespassing on Slopes	NONE
Sloughing or Erosion of Slopes or Abutments	SOME MINOR SLOUGHING OF D/S SLOPE - CONCEALED BY HEAVY GRASS
Rock Slope Protection-Riprap Failures	SOME HAND PLACED RIPRAP ON U/S SLOPE IS DISPLACED. VERY MINOR
Unusual Movement or Cracking at or Near Toes	NONE OBSERVED
Unusual Embankment or Downstream Seepage	SOME AT RT. & LFT D/S TOE OF DAM. OBSERVATION WELL AT RT. D/S TOE W/ ARTESIAN CONDITION.
Piping or Boils	NONE OBSERVED
Foundation Drainage Features	OBSERVATION WELL AT RT. D/S TOE IS 2 1/2" I.D. PIPE DISCHARGING AT 0.04 gal./min.
Toe Drains	POSSIBLE DRAIN WHERE STONES DISCOVERED @ LEFT D/S TOE
Instrumentation System	NONE OBSERVED



# PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT SHUTTLEMEADOW RES. DAM

DATE 12/6/78

PROJECT FEATURE DIVERSION CHANNELS INTO RES. BY PMH, CEG, GL, TK

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE</u>	2 DIVERSIONS INTO RESERVOIR, 1 @ EACH END OF DAM.
a) <u>Approach Channel</u>	
Slope Conditions	SIDE OF CHANNEL IS CONCRETE
Bottom Conditions	HAND PLACED STONE
Rock Slides or Falls	NONE
Log Boom	NONE
Debris	NONE
Condition of Concrete Lining	CONCRETE SIDES HAVE SOME CRACKING AND SPALLING.
Drains or Weep Holes	NONE OBSERVED
b) <u>Intake Structure</u>	
Condition of Concrete	IVA
Stop Logs and Slots	4' HIGH WOODEN STOPLOGS-APPEARED TO BE IN GOOD CONDITION. LOCATED IN SPILLWAY CHANNEL.

# PERIODIC INSPECTION CHECK LIST

Page A-4

PROJECT SHUTTLE MEADOW RES. DAM

DATE 12/6/78

PROJECT FEATURE OUTLET CONTROL TOWER

BY PMH, CRG, GC, TK

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-CONTROL TOWER</u>	
a) <u>Concrete and Structural</u>	
General Condition	BRICK CONSTRUCTION ABOVE WATER CONCRETE BELOW
Condition of Joints	TOWER OUT IN REPAIR - NOT ACCESSIBLE TO INSPECTION
Spalling	NOT OBSERVED
Visible Reinforcing	SOME NOTED ON OUTSIDE
Rusting or Staining of Concrete	NONE OBSERVED
Any Seepage or Efflorescence	NOT OBSERVED
Joint Alignment	SOME
Unusual Seepage or Leaks in Gate Chamber	NOT OBSERVED
Cracks	NOT OBSERVED
Rusting or Corrosion of Steel	NONE OBSERVED
b) <u>Mechanical and Electrical</u>	NOT OBSERVED
Air Vents	NA
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	
Service Gates	VALVE STEMS REMOVED. SOME GATES INOPERABLE
Emergency Gates	NONE
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

## PERIODIC INSPECTION CHECK LIST

Page A-5PROJECT SHUTTLEMEADOW RES. DAMDATE 12/6/78PROJECT FEATURE LOW LEVEL PIPES/CONDUITS BY PMH, CRG, GC, TK

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-TRANSITION AND CONDUIT</u> <del>General Condition of Concrete</del> <del>Rust or Staining on Concrete</del> <del>Spalling</del> <del>Erosion or Cavitation</del> <del>Cracking</del> <del>Alignment of Monoliths</del> <del>Alignment of Joints</del> <del>Numbering of Monoliths</del>	LOW LEVEL OUTLETS: PIPES/CONDUITS UNDER DAM NOT ABLE TO OBSERVE CONDUITS, HOWEVER, ACCORDING TO THE OWNER'S CARETAKER, SOME LOW LEVEL OUTLETS ARE OPERABLE.  MANHOLES AND COVERED, BRICK-LINED HOLES WERE DISCOVERED D/S OF DAM. COVERS IN VERY POOR SHAPE. OUTLETS TO PUMP STATION D/S OF DAM

# PERIODIC INSPECTION CHECK LIST

Page A-6

PROJECT SHUTTLEMEADOW RES. DAM

DATE 12/6/78

PROJECT FEATURE SPILLWAY AND CHANNELS

BY PMH, CRG, GC, TK

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	
a) <u>Approach Channel</u>	
General Condition	GOOD. STONE AND MORTAR, W/ HAND PLACED STONE ON BOTTOM OF CHANNEL
Loose Rock Overhanging Channel	NONE
Trees Overhanging Channel	NONE
Floor of Approach Channel	HAND PLACED STONE. SOME GRASS. GOOD CONDITION
b) <u>Weir and Training Walls</u>	
General Condition of Concrete	CONCRETE AND STONE. GOOD CONDITION.
Rust or Staining	NONE OBSERVED.
Spalling	NONE
Any Visible Reinforcing	NONE
Any Seepage or Efflorescence	EFFLORESCENCE FROM MORTAR ON SIDE STONE WALLS.
Drain Holes	NONE OBSERVED
c) <u>Discharge Channel</u>	
General Condition	GOOD
Loose Rock Overhanging Channel	NONE
Trees Overhanging Channel	SOME TREES
Floor of Channel	HAND PLACED STONE. CONCRETE SPILLWAY CAP
Other Obstructions	IN APPROACH CHANNEL: 4' HIGH WOOD STOP LOGS STONE BLOCK DISSIPATORS @ RIGHT ANGLE BEND IN CHANNEL BEFORE SPILLWAY CREST.

# PERIODIC INSPECTION CHECK LIST

Page A-7

PROJECT SHUTTLEMEADOW RES. DAM

DATE 12/6/78

PROJECT FEATURE CONCRETE BRIDGES

BY PMH, CRG

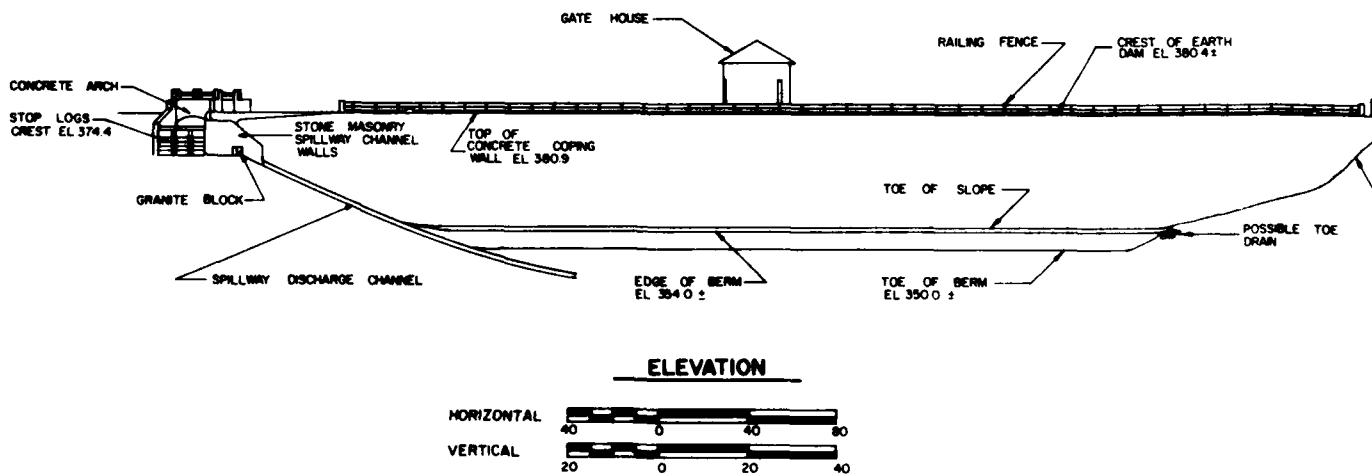
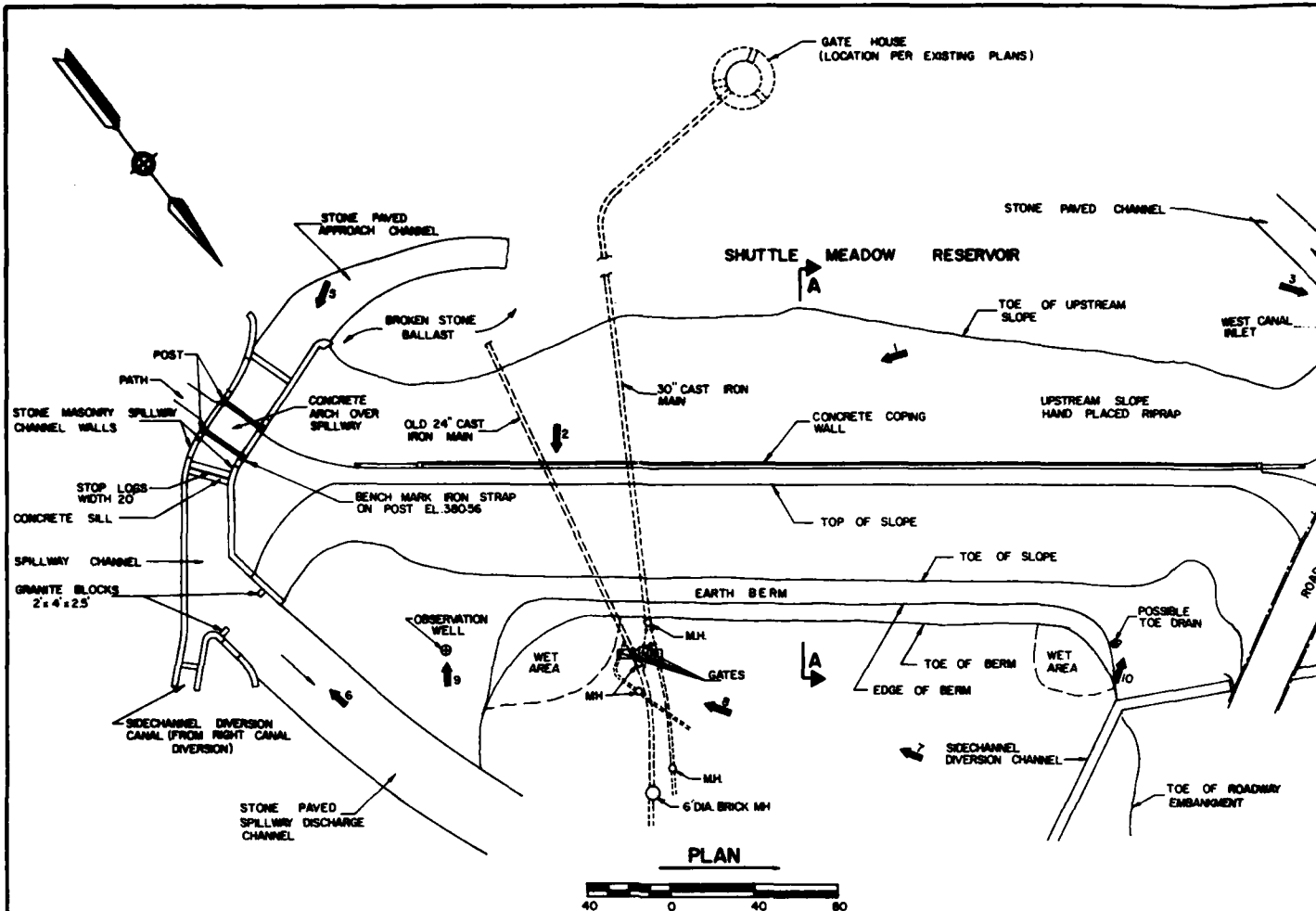
AREA EVALUATED	CONDITION
<u>OUTLET WORKS-SERVICE BRIDGE</u>	<u>BRIDGES OVER SPILLWAY AT RIGHT END AND OVER DIVERSION INLET AT LEFT END OF DAM.</u>
a) <u>Super Structure</u>	
Bearings	NA
Anchor Bolts	NA
Bridge Seat	FOUNDED ON STEEL RAIL CHANNEL LINING
Longitudinal Members	
Under Side of Deck	CONCRETE ARCHES AT BOTH ENDS
Secondary Bracing	NA
Deck	GOOD CONDITION
Drainage System	NONE CHECKED
Railings	METAL TUBULAR
Expansion Joints	GOOD CONDITION. SOME FILLING REQUIRED.
Paint	NONE
b) <u>Abutment &amp; Piers</u>	
General Condition of Concrete	GOOD. SOME CRACKING AT LEFT ROADWAY BRIDGE OVER DIVERSION
Alignment of Abutment	AT LEFT END: CRACKS SET SET INTO 1.2 INCHES.
Approach to Bridge	GRAVELLED EARTH ON RIGHT END. PAVED ROAD IN GOOD CONDITION AT LEFT.
Condition of Seat & Backwall	NA

APPENDIX  
SECTION B: EXISTING DATA

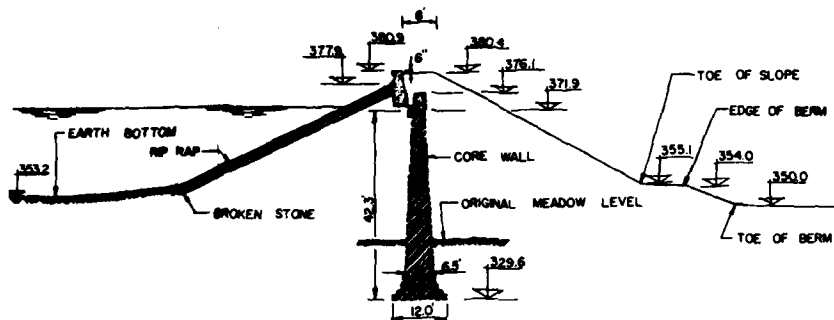
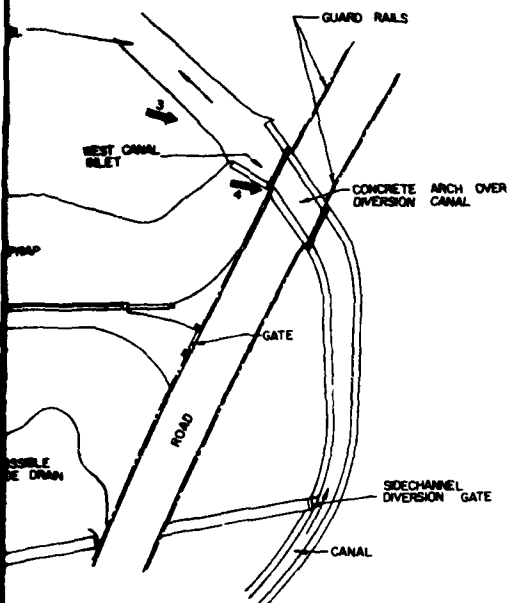
## APPENDIX

### SECTION B: EXISTING DATA SHUTTLE MEADOW RESERVOIR DAM

	<u>Page</u>
Dam Plan, Profile and Sections.....	B-1
List of Existing Plans.....	B-2
Summary of Data and Correspondence.....	B-3, B-4
Data and Correspondence.....	B-5 to B-36







SECTION A-A



### NOTES

1 THIS PLAN WAS COMPILED FROM EXISTING PLANS FOR THE 1912 RAISING OF THE DAM AND THE 1938 REPAIRS TO THE DAM AS DRAWN BY THE CITY OF NEW BRITAIN ENGINEERING DEPARTMENT. EXISTING INFORMATION WAS SUPPLEMENTED BY A ROUGH FIELD SURVEY BY CAHN ENGINEERS. NOT ALL TOPOGRAPHIC AND/OR STRUCTURAL FEATURES ARE NECESSARILY IDENTIFIED ON THIS PLAN.

2 ALL ELEVATIONS ARE MEAN SEA LEVEL DATUM TAKEN OFF OF EXISTING PLANS AND FROM FIELD SURVEY REFERENCED TO A BENCH MARK ON THE DAM WHICH WAS IDENTIFIED BY THE CITY OF NEW BRITAIN ENGINEERING DEPARTMENT.

3 PHOTO NUMBER AND DIRECTION

CAHN ENGINEERS INC WALLINGFORD, CONNECTICUT ENGINEER		U.S. ARMY ENGINEER DIV NEW ENGLAND CORP OF ENGINEERS WALTHAM, MASS	
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS			
<b>SHUTTLE MEADOW RESERVOIR DAM</b>			
WILLOW BROOK		NEW BRITAIN, SOUTHINGTON, CONNECTICUT	
DRAWN BY	CHECKED BY	APPROVED BY	SCALE AS NOTED
M.N.	CRG	PMH	DATE MARCH, 1979 PAGE 8-1

SHUTTLE MEADOW RESERVOIR DAM

LIST OF EXISTING PLANS

City of New Britain, Water Works  
"Raising of Shuttle Meadow Dam"  
Plan and Cross Section of Raising  
City Engineering Department  
1909

Shuttle Meadow Dam  
"Sections and Details of Dam"  
Board of Water Commissioners  
City of New Britian  
J. W. Holden, Chief Engineer  
Oct. 4, 1938

# SUMMARY OF DATA AND CORRESPONDENCE

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# SUMMARY OF DATA AND CORRESPONDENCE

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Aug. 12 1963	George W. Wood	B. H. Palmer Chandler and Palmer Civil Engineers	Description of wet areas at downstream toe of dam	B-34
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STATE BOARD FOR THE SUPERVISION OF DAMS  
INVENTORY DATA

21

CT - 162

NAME OF DAM OR POND Little New Britain

CODE NO. M B B W 6-8

LOCATION OF STRUCTURE:

Town New Britain and Southington

Name of Stream \_\_\_\_\_

U.S.G.S. Quad. New Britain Long. 72 19.1 Lat. 41-38.7

OWNER: New Britain Water Co.

Address New Britain

Telephone \_\_\_\_\_

04  
1113

Pond Used For: Drinking water.

Dimensions of Pond: Width 1/4 Mi. Length 1 Mile Area 205. A

Depth of Water below Spillway Level (Downstream) 24'

Total Length of Dam 600 ± Length of Spillway 20

Height of Abutments above Spillway 6'

Type of Spillway Construction Concrete

Type of Dike Construction Earth

Downstream Conditions steep area to built up area.

Summary of File Data \_\_\_\_\_

Remarks This pond and dam is in both N. B. Britain and Southington. Most is in Southington so it is classed as Southington. Also referenced under N. B. This is a project of some importance. Should be given to R. M. in this District.

B-5

857  
Rebuilt  
1894

DESCRIPTION OF THE NEW DAM  
OF THE  
NEW BRITAIN WATER WORKS RESERVOIR.

by Percy M. Blake,

chief engineer, Hyde Park, Mass.

Read at the Spring Meeting Connecticut Association of  
Civil Engineers and Surveyors, New Britain, May 31, 1893.

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For the conception and successful execution of public improvements the engineer is responsible, and early in his experience he is made to see and feel that a poor piece of municipal work is neither forgotten nor forgiven by the observing and critical public. Exceed appropriations and estimates he may, but this critical public will demand of him a reasonable degree of excellence in all his undertakings, and if he fails to attain this, then no financial economy can save him.

The municipal improvement which you are invited to examine today and for the designing and execution of which the writer is to a large degree responsible, is not an extraordinary nor an unusual one. It is but one of a class which is bound to become necessary in the growth of any New England city.

New Britain was among the pioneers in introducing a public water supply, water having been introduced from the Shuttle Meadow water-shed in 1857 when the borough had a population of less than 4,500 people. At that time there were few public water supplies in the New England states. An ordinary earthen dam of moderate height and simple design was constructed across the valley of Shuttle Meadow Brook, and an artificial lake of some 160 acres made by impounding the water gathered from a water-shed area of 619 acres. In 1890 with a population of 19,000, the city of New Britain had utilized nearly to its fullest capacity its sources of supply supplemented as it had been in 1884 with a secondary water-shed area of 446 acres. The records of previous years showed to a committee in 1890 that much water from these watershed areas had been lost in some years for want of adequate storage capacity in the reservoir or lake formed by the dam in 1857. An examination of all the facts in the case demonstrated the necessity for a new and higher dam and the feasibility of annexing another additional watershed area of 476 acres. In 1891, the city authorized the improvement of the public water supply by the construction of a new dam and gatehouse, the laying of a large leading main from this new gatehouse to the city, and the addition of the new watershed area by the construction of an intercepting canal discharging into the Shuttle Meadow Lake through a channel at the westerly end of the new dam. B-6

There are no specially unique features about the dam, the canal or gatehouse, but it was recognized that in the construction of the dam, absolute stability and strength to retain nearly a billion and a quarter gallons of water must be provided, that the water slope of the structure

structure must be adequately protected against erosion by wave action and ice, and that the joint between the bottom of the structure and its natural foundation must be so tight as to admit of no actual flow of water through it.

The dam is very nearly 600 feet long and 20 feet wide on its top, about 33 feet high, with inner slope two on one and outer slope one and eight-tenths on one, and contains about 38,000 cubic yards of embankment. Its cost will be, when completed, very nearly \$51,000. As is common in all properly designed structures of this kind, there is a core wall running through the axis of the dam. The wall consists ~~entirely~~ entirely of hand-made concrete resting on a bed foundation of hard compact material, generally of a gravelly nature, but very nearly water tight, and for a limited portion of the way, on bed rock of ledge of the same composition as that to be seen in the abrupt hillside against which the west end of the dam abuts. This hand-made concrete was built up between mould boards by hand, the matrix consisting of a strong cement mortar, and the filling being angular fragments of stone varying in dimensions from 3 to 10 inches in greatest length. These stones were firmly and individually bedded with a mason's small hammer in the stiff matrix until the latter flushed to the surface under the blow and submerged the stone. Frequent examinations and tests of the composition were made by digging into and breaking up portions which had become set and partially hardened, and in no instance did such examination show that the mass of the wall was other than a close water-tight structure. The writer's experience with core wall and similar work has been such as to demonstrate that this form of concrete is superior in its strength and homogeneity to any mixed concrete which can be made and applied in any progressive building. The amount of core wall masonry of this class in the dam is 2,800 cubic yards, and the price paid per yard was \$6, which in this case afforded a handsome profit to the contractor.

The form and superficial finish of the dam can be better understood by the examination which you will make of it than by any verbal description. Such curves and lines in a structure of this kind as lie outside the shape and dimensions necessary to give the required stability and strength, are designed mainly to please the eye, and, on the inner slope of the dam, to resist the wave action and the grinding force of floating ice.

The gatehouse is cylindrical, with admission ports at different elevations and ample internal screening capacity. The internal diameter of the superstructure or wheel-room is 21 feet, and the height of wall in this room is 16 feet, giving ample space overhead for the play of the screen hoisting apparatus. The screens are in square sections and are in all respects interchangeable, so that no inconvenience can be experienced in removing and replacing them. The admission ports are controlled by heavy sluice gates located within the substructure and protected on the outside by an iron shield, which serves to some extent as a very coarse strainer, and prevents the admission into the gatehouse of large floating objects. As in the case of the dam, an examination of this structure will convey a much clearer idea of its details than a description at this time. From this gatehouse a 30 inch heavy cast-iron main is laid under the dam to a point in the meadow below, where by convenient connections controlled by suitable gates, the water is delivered

into the 24 inch cast-iron leading main which supplies the city with water. There is also a 24 inch cast iron main laid under the dam to the old gatehouse, which it is proposed to retain and use as an alternative outlet if occasion requires, and this 24 inch main joins by independent connection, the new 24 inch main just referred to. Water can be drawn through either gatehouse by this plan.

The waste-way over which any surplus water yielded by the three watersheds will pass from the lake, is 30 feet in width, and consists of a cut stone sill and apron, with paved channel above and below. It is estimated that at no time will the depth of water over this waste-way be greater than seven inches.

As an aid in preserving the local purity of the water in the vicinity of the dam, the bottom and slopes of the reservoir have been ballasted with broken stone and small boulders, this work having in view as an important object, the preventing of turbidity of the water in the immediate vicinity of the gatehouse, which might be caused by the wind wash on an unprotected and rather soft shore line.

In constructing this dam and gatehouse, the original dam was entirely removed and such of the material forming it as was found suitable for the purpose was deposited in the embankment of the new dam, and the space formerly occupied by the old dam is now a very level reservoir bottom just within the flow lines of the new structure.

The watershed area of 476 acres recently added, has been made available by the construction of a canal nearly 8,000 feet in length. The grade of this canal is one foot in 1,000 and the discharging capacity has been based upon a cross section having a bottom width of five feet and slopes on one-half on one. The shortest radius used in the curves employed in locating this canal is 100 feet, and the maximum depth of water which has thus far been observed, and it was an extreme case, is five feet and three inches. The design for this canal includes the construction in its outer bank of four waste weirs, the available level of which is to be placed five feet above the bed of the canal. These waste-weirs will act as relief valves, and in case of the unexpected backing of ice, or the appearance of other obstacles, the section or sections of the canal above the point of obstruction will be protected from an overflow of the canal bank with its attendant danger.

This canal is yet in an unfinished condition, although comparatively little remains to be done to it. The effects produced by ice last winter and by the varying and sometimes excessive flow of water during the spring months, have been carefully noted and the work of applying a permanent protection at the points needing it has been begun and will in a comparatively short time be complete.

This protection will consist of paving in the form of close bedded work or an apron work, ballasting, benching, and in one or two localities evening and tightening with concrete. The cost of this canal when completed will not vary much from \$40,000. The function of this item of improvement is to divert and convey the waters of three vigorous brooks and to collect in addition thereto, such surface water as formerly found its way over the natural slopes of the watershed unintercepted, to the lower land.



A canal of this kind, in which the water level will vary greatly, and which follows also tortuous a course, will undoubtedly require regular inspection and frequent repairs and corrections. It is not practicable or expedient to apply a continuous form of protection for the entire length of the canal, owing to the disproportionate cost which would be entailed thereby. Monthly inspections of the condition of the canal will show that the looser material washed into the bottom will require removal in order to restore the regular grade of the canal and insure a uniform delivering capacity. In very cold and dry winters the banks will be eroded by frost and ics, and it will be found necessary to add additional protection occasionally, or to replace that displaced by such ravages.

The following memorandum from the report recently made to the Board of Water Commissioners, will be of interest:

"When the West Canal is completed, the city will be in possession of the following sources of supply:

First, the Shuttle Meadow Reservoir, with a high water surface of about 185 acres and amximum storage capacity when full to the level of the new waste-way, of more than 1,000,000,000 gallons.

Second, the lake watershed (net) of 599 acres.

Third, the Panther Swamp watershed of 446 acres.

Fourth, the West Canal watershed of 476 acres.

Total watershed area 1,521 acres.

This area is 8  $\frac{2}{9}$  times the new high water area of the reservoir.

Taking into account the saving of the water which the records show has been wasted frequently in the past for want of storage capacity, and assuming that no undue waste of water by the many users thereof is permitted, the watershed area now contributing to the reservoir will supply water enough for a population of at least 30,000 people."

The productiveness of this combined watershed area is shown by its yield of water since January 1893, at which time the water in the reservoir stood at an elevation of ten feet below the overflow level of the original dam and twenty feet below the waste way of the new dam. On the 15th day of May the water had risen from the lowest point just named to elevation 54, and gain of 14 feet. Yet so varying is the yield of a given watershed that it is not possible to formulate any rule, even with all the data relating to the amount and distribution of the rainfall, drainage area, storage capacity, and draft in hand, by which the amount of water annually available for use above the outlet level of the reservoir can be determined with precision. These combined sources of New Britain are, however, so related to the reservoir that it will be found, if accurate records are kept hereafter, that the range of fluctuation in Shuttle Meadow reservoir will be kept within extremes nearer together than those observed in the past.

As a natural and desirable sequel to the improvements thus far made, it is believed that a cleaning of the shores of the reservoir between

between the plains of high and low water will result in an appreciable improvement in the average quality of the water. The removal of all soluble soils and vegetation around the lake and the protection of the steeper slopes by stone ballasting, are the principal items in this particular line.

Nothing has contributed more to the successful conduct of the improvements described above than the unanimous and hearty support on the part of the Board of Water Commissioners, reinforced, it is believed by public opinion and certainly by official public action. From the inception of the work at the hands of the committee in 1891 no interference with the programme of work has been made, although the earlier portions of the actual construction were not proceeded with as rapidly as they might have been. When the project is considered in its entirety, it is easily seen that no item of work undertaken could have been omitted.

The dam and canal together with the waste way and approaches and the ballasting of the reservoir basin were done by contract with the Troy Public Works Co., of Troy, New York. The Specifications for all of this work, and which became a part of the contract upon the execution of the latter, were rigidly drawn, and to no other part of the arrangement is more credit due for the excellent results reached, than the efforts constantly put forth by the officials and foremen of this Company. It is not my custom to issue letters of recommendation which might be interpreted as indicating a decided partiality for one contractor to the exclusion of others, as current events are constantly showing the small value which recommendations, honestly given in some instances have. So I will <sup>not</sup> at this time express any further appreciation of the work done by the Troy Company. You will shortly have ample opportunity to judge for yourselves of the competency of the work. If you find flaws, or ought to criticise, you are respectfully referred to Mr. Richard W. Sherman, the President of the Troy Public Works Co., who is present with us to-day to face the verdict. In his company I can also single out Mr. Charles H. Eglee, who, as the contractor, laid the new 24 inch main through which the improved liquid will find its way to the citizens of New Britain, and who may be able to explain to you why his pipes are tight instead of leaky. From what has been said you may perhaps infer that neither of these contractors are yet upon the black list for poor work.

Only a limited portion of the work done in improving the water supply of New Britain as above outlined has been day-labor, so called, as it has been the belief of the Commissioners and certainly of the writer, that day-labor is too costly a method to be enjoyed at the public expense. There are in all large engineering projects certain minor and irregular portions of the work which can best be done by day-labor, but I believe that a competent and experienced contractor can so organize and manage his working forces, tools, machinery and supplies that he can conduct the operations of construction under a clearly drawn contract to better advantage and at less cost to the municipality which may elect to employ him. This view has been held throughout by the Water Commissioners in this case, and the financial results have fully justified their judgement as being correct.

I wish to improve this opportunity by referring to the connection of Mr. Arthur W. Rice, C.E. of New Britain, a member of your Society, with this project. He made the surveys upon which the plans were based and faithfully and ~~painstakingly~~ in a painstaking way superintended the construction of the embankment of the dam and the superstructure of the gate house. He spared neither time nor inclination, and to him credit is due for the stability of these structures, and inquiries concerning the details of this portion of the work should be addressed to him.

As a proper supplement to the description given we have photographs of the work taken at various stages of its progress. The work is about two weeks behind schedule time. I prefer to look at engineering work before it is entirely covered up. It is far enough along so that you can get an idea of the methods employed.

The work of construction was begun November 8, 1891, and continued till January 20, 1892. It was resumed April 7, 1892 and again suspended January 5, 1893. It was again resumed April 12, 1893 and will probably be completed about June 15, 1893.

Some discussion followed, in which Messrs Bunce, Chandler, Loomis and Blake participated.

Copied in the office of the  
Secretary, Conn. Soc. C.E.  
from bound copy of the  
Proceedings of 1894.  
Nov. 28, 1946.

Hartford, Conn.  
Oct. 7, 1938

Mr. W. H. Cadwell, Chairman  
Conn. Board of Civil Engineers

Re: Safety of Shuttle Meadow Dam

The following information has been collected on the above question. Perhaps others have data differing from the matter here presented. It would be appreciated if any mistakes were pointed out to me.

1. Herewith is the meriden sheet of the state topographical map. I walked over about one mile of each canal trying to identify their locations but found the map is quite different from the actual ground, so I am making no attempt to locate the canals on the map.

2. Four photographs taken by me accompany this communication.

- (1) General view of downstream slope showing areas washed out by spray.
- (2) General view of south end of dam.
- (3) Details of spillway.
- (4) General view of south end of dam.

3. In the 1894 Proceedings of the Conn. Soc. of C.E., pages 30-44, is a paper on the Shuttle Meadow Dam and reservoir by Percy M. Blake who designed and supervised the work. The following is taken from this paper.

"There was a dam and reservoir built in about 1857 which served as a water supply before the later work was authorized. So, the topography of the contour map was taken in 1889-1890. This early dam and reservoir are the ones shown on the map. The new dam was authorized in 1891 and completed two years later.

"In constructing this dam and gate house the original dam was entirely removed."

"The dam is very nearly 600 ft. long and 20 ft. wide on top, about 10 ft. high, with inner slope two on one and outer slope one and eight on one "XXX". There is a core wall running through the axis of the dam. This hand made concrete was built up between mould boards by hand. The matrix consisting of a strong cement mortar and the filling being angular fragments of stone varying in dimensions from 3 to 10 inches in greatest length. These stones were firmly and individually bedded with a mason's small hammer in the stiff matrix until the latter flushed

to the surface under its blows."

T. H. McKenzie, state engineer, who inspected the work stated in discussing the paper that "the core wall is one of the best water tight walls that I have seen." He said that the inner slope of the dam was protected by a paving while the outer slope was turfed.

Mr. Blake said, "the waste-way XXX is 30 ft. in width and consists of a cut some sill and apron with paved channel above and below." "At no time will the depth of water over this waste-way be greater than seven inches." (Photos No. 3 and No. 4 show that the present waste-way is not like the one Blake built.)

Mr. Blake recites that the reservoir will receive the run-off from the following areas.

185	acres	of	reservoir	surface
599	"	"	watershed	above the dam exclusive of the first item.
446	"	"	Panther Swamp watershed	through the canal entering the reservoir from the south. See photo No. 4
476	"	"	"West Canal" watershed.	(The map shows that this canal should be called the "North" Canal.

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1706 acres = 2.66 square miles.

In the discussion it was brought out that the "F.O. Norton" brand of Rosendale cement was used in building the core wall, also that the latter was  $8\frac{1}{2}$  ft. wide just above the base and 30 inches at the top, and its top was one foot above the water surface when the reservoir was full. Mr. Blake also said the thickness of the dam at the wash-line is 38 feet and the top of the embankment was 6 ft. above high water mark.

4. In addition to the canals mentioned above the Shuttle Meadow reservoir receives water from the Wolcott Reservoir located about 5 or 6 miles west of Shuttle Meadow through two 20 inch pipe lines that join and discharge in to Shuttle Meadow reservoir through a short length of 30 inch pipe. Wolcott is about 390 ft. higher than Shuttle Meadow so the discharge through the 29,500 ft. of the "two 20 inch" main is controlled by opening valves at the ~~discharge~~ <sup>outlet</sup> end.

5. Mr. Holden, Engineer of New Britain Water Dept. gave me a paper on which were listed the areas of the different watershed supplying the city with water. From this the following is taken:

Above Shuttle Meadow Dam	749.95	A.	=	1.17	sq. mi.
East Canal (Panther Swamp)	450.16	"	=	0.70	" "
West Canal (I call it	681.60	"	=	1.07	" "
North)					
	1881.71	A	=	2.94	sq. mi.

The above agrees with Mr. Blake's figures as the North Canal has been extended since it was first built so more area drains into it.

6. I looked over each of the canals and found them very irregular in cross section. They are not paved except at the discharge ends at the dam. The Panther Swamp canal seems a little deeper but not so wide as the other. We would not be very far from correct should we consider each canal to have a discharge capacity equal to a canal having 6 ft. of bottom width with side slopes of  $1\frac{1}{2}$  to 1 and running 5 ft. deep with a canal slope of 1 ft. in 1000 ft. However, the full watershed run off is available to the city and the canals can easily be made large enough to get all of it so we should consider the full area when computing the spillway requirements.

7. The dam was raised in 1912 and the spillway has been changed in character since Percy Blake built it. The coping wall is a new feature not in the 1891 design. A test pit was dug south of the stairway to uncover the top of the core wall also the bottom of the coping wall. The spot is marked (A) on photo 1. A similar test pit was dug at the point marked (B.). Each of these pits was opposite the center of a "slide". On the back of No. 2 picture I have placed dimensions as I took them in a very general way when inspecting the holes. They check very well with the drawings prepared by Mr. Holden showing cross sections of the dam. This drawing is attached. There is no evidence in the test pits of any "piping" of water over the top of the present core wall, but there is evidence of saturated earth near the bottom of the coping wall foundation and in the area between the coping wall and the core wall. Quite a bit of the earth excavated from these pits shows it was very soggy with water when shoveled out.

8. Mr. Lawrence, caretaker at the reservoir told me the maximum height of the water surface on Sept. 21, 1938 as measured by him at the gauge at the stilling basin shed was about 6 inches above the wrought iron strap fastened to the masonry. The elevation of the sharp edge of this iron strap is 376.02 said Mr. Holden when he called on me Oct. 7, 1938. If 0.48 ft. is added to this the maximum water height on Sept. 21, 1938 is established at elevation 376.5 ft. This figure agrees closely with  $4\frac{1}{2}$  ft. below the top of the concrete coping wall.

9. Mr. Holden told me that he had removed some of the slope paving stones near the coping wall and found the stones were resting on a foundation bed of small stones or gravel. My own inspection of the coping wall and slope paving shows that there has been no apparent settlement of either since they were placed in 1912. All lines and surfaces appear true at the present time.

10. Here are some elevations:

380.92 Top of concrete coping.

0.75

380.17 Top of earthen dam

374.20 Top of fixed spillway

6.00 Kneeboard with no water going over spillway

B-14

380.17 Top of earthen dam  
376.50 Water surface, Sept. 21, 1938  
3.67 Freeboard, " " "

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376.50 Water Surface, Sept. 21, 1938  
375.84 Top of present core wall.  
0.66

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376.50 Water surface Sept. 21, 1938  
374.20 Top of fixed spillway  
2.30

11. Attached is a photostat of plottings of a large number of actual runoffs of magnitude in the northeastern states and elsewhere. It will be sometime before the actual run-offs from the recent heavy rains are available, as the last storm was very severe it is to be expected that some new records for small watersheds were made.

Yours truly,

(Signed) Edward W. Bush

89-1-1

Preliminary Study of Safety Requirements

of Shuttle Meadow Dam

By Edward W. Bush  
October 8, 1938

from Mr. Cadwell  
T. J. C.

-----  
(Confidential for Board Members only);

1. Present spillway capacity.

Photo 3. shows the spillway is divided into three parts each about 6 ft. long on the crest. The walk way cuts off the effectiveness of the weir so that the maximum discharge height is only about 24 inches. The computation is, therefore, made on that basis.

According to the 1911 American Pocket Book the table of weir discharges gives for  $h = 2$  ft. 10.58 c.f.s. per ft. of weir; but there are 6 used contractions equivalent to shortening the length of the weir by  $6 \times \frac{2 \text{ ft. head}}{10} = 1.2$  ft.

The weir length, therefore, is  $18 - 1.2 = 16.8$  ft. and the total discharge capacity of the present weir is  $10.58 \times 16.8 = 178$  c.f.s. a ridiculously small amount for a watershed of 2.94 sq. miles. The original Blake spillway had a discharge capacity three or four times the present one and the present drainage area is larger than the area considered by Blake.

I dislike the present form of spillway as I believe it invites the insertion of extra 6 inch planks as temporary flash boards. The approach channel is very shallow and constricted and it could easily be blocked with floating ice or debris. A masonry weir with a rounded crest is much better and it should not have pipe holes or wooden accessories installed to which flash boards could be attached.

Mr. Blake had a 30 ft. weir built and his idea of a longer spillway without depending in a considerable depth of discharge is much better than a shorter weir and a greater depth. His weir length however is much too short as later flood statistics have led engineers to provide more ample spillways. Records have been broken every few years as more engineers are interested in recording the unusual water heights.

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2. Our Board should "lean over backwards" when fixing any requirements to insure the safety of a dam located just above a large section of New Britain where the results would be appalling should the dam go out.

No photostat chart shows many specific records of run-offs that would justify our estimating it would not be unreasonable to consider a maximum run-off of 2000 c.f.s. per sq. mile. These records are, I believe, intense rain falls for rather limited periods. The



Shuttle Mound reservoir is about 200 acres in extent and it will require quite a bit of hard rainfall to raise its surface one foot. I know of no records or formula that would unerringly guide us to the correct answer. It seems to be a matter of judgment formed after a study of the maximum storms and run-offs. However, no longer can it be said that Connecticut will never have the large rainfalls that might be possible farther north because we have just had a 13 inch storm and this is about as big as any that ever fell elsewhere in New England. We, therefore, are subject to any intensities that might be found at any other place in the northeastern part of the country.

Here are some records of large run-offs in terms of cubic feet per second per square mile of watershed found in Mr. Jarvis' paper.

<u>cu. ft. per sec. per square mile</u>	<u>watershed sq. mi.</u>	<u>date</u>	<u>Location</u>
2300	1.5	May, 1916	Mad Creek, Leroy, N.Y.
1595	1.7	July, 1914	Green Branch, Bridgeville, Pa.
930	2.1	-	Indian Run, Letort, Pa.
630	2.2	July, 1914	Canadochly Br. E. Prospect, Pa.
480	2.7	July, 1912	Calvin Run, Grindston, Pa.
1000	3.5	July, 1918	Honey Creek, New Carlisle, Ohio
500	6.0	July, 1903	Brush Creek, Jeanette, Pa.
400	8.0	May, 1894	Burgeon's Run, Pa.
241	9.4	Sep. 1906	Mill Brook, Edmeston, Pa.

The smaller the watershed the greater the likelihood of higher figures so we could translate the last three records in to what might have occurred on a 2.94 sq. mi. watershed by using the diagonal lines on the photostat chart. Then the last three respectively become 700, 625 and 400 c.f.s. per sq. mile.

### 3. New Spillway Capacity.

To start the discussion I will assume the spillway should be able to pass a flow of 800 c.f.s. per sq. mile. This multiplied by 2.94 square miles = 1470 c.f.s. discharge. Divide this by a 90 ft. spillway requires each foot of weir to pass 16.33 c.f.s. and a height of water running over the weir of about 2.5 ft. would do the trick.

The freeboard or difference in elevation between the top of the spillway and the top of the dam should be increased over the skimpy 6 ft. now used. I would prefer 10 ft. and would not accept less than 8 ft.

The present top width of dam is 9 ft. but originally the width was 20 ft. The latter should be the minimum. Also the top surface should be higher along the down stream edge and the surface

should be paved with either concrete or a macadam road well set with asphaltic oil. The tops of the canal dikes should be fully as high and as safe as the main dam.

6. If the down stream toe of the earthen dam is soggy, blind drains should be built and perhaps a rip rap toe inserted.

(signed) E. W. Bush, Member  
State Board of Civil Engineers.

EWB/C.

3 copies made 4/7/42

1 - Mr. Palmer

1 - Mr. Blair

84-1-1

from Mr. Andrews of

New Britain, Conn.  
December 19, 1938

Board of Water Commissioners  
City of New Britain  
New Britain, Conn.

RE: REPAIRS TO SHUTTLE MEADOW DAM

Gentlemen:

As you are aware the Connecticut Board of Civil Engineers received a petition shortly after the heavy storm of September 21, 1938 signed by about twenty persons who might suffer loss or damage by the breaking away of the Shuttle Meadow Dam, asking our Board to investigate the safety of this dam all as prescribed in Section 3058 of the General Statutes of 1918.

After an inspection and study by the four members signing this communication, to whom the matter was referred, a decision was reached that the dam at the time of the inspection on September 29, 1938 was not safe if a very hard storm and wind came, and the Board sent you a letter dated October 12, 1938 stating we thought you should not raise the water level in the reservoir above elevation 372 (your datum) until certain permanent changes were made.

We now direct you to make the following changes, or repairs, before raising the water level above elevation 372 referred to above:

1. We consider the present spillway inadequate and of poor design. We direct that the steel work and small bridge be removed and that a permanent masonry discharge weir with a rounded top having no places for flash boards be installed instead of the present spillway. The top of the new masonry weir to be no higher than the top of the weir which is at elevation 374 ft. above your datum.

In addition to the above there shall be another masonry spillway built that is not less than 50 ft. long and with a top elevation of 374, with no provisions for flash boards.

If you prefer one spillway having a length of 70 ft. instead of the two spillways mentioned above, you may submit plans for our approval.

B-19

Any spillway shall have an adequate approach channel of such design that there is little likelihood of its becoming clogged by debris or floating ice. An adequate discharge channel shall lead from any spillway of such shape and with such bottom paving and side walls as will cause no scour or damage to the toe of the earthen dam when the spillway is discharging at its full capacity.

2. We forbid the use of flash boards at the Shuttle Meadow Dam.
3. We direct you to widen the top of the dam to 20 ft. exclusive of the 24 in. coping, also to raise the top so the down stream edge of the top will be elevation 382 and about one (1) ft. above the top of the coping, and so the top of the dam slopes downward to said coping top. The down stream slope of the widened dam shall be not less than one vertical on two horizontal with a flat space of not less than 7 ft. having a drop of one (1) ft. in the seven (7) ft. horizontal located about one half way down the back slope. All the present sod and top loam shall be removed before the new filling is placed, and afterwards the new parts shall be loamed, seeded, or turfed..

The present toe shall be explored by excavating test trenches to see if the soil is soggy with water, and we wish to inspect these trenches. If soggy earth is found blind drains or trenches shall be excavated and filled with stones or tile drains must be installed to drain the present toe before the new filling is added.

The lower part of the new filling shall consist of a toe fill of dumped stone or paving not less than 5 ft. high with side slopes of 1 vertical to  $1\frac{1}{2}$  horizontal. Blind drains shall lead through the new earth filling to this toe of stone filling.

4. The masonry core wall shall be extended upward to elevation 380, ~~dam~~ with special care being taken to bind the new masonry with the old, said additional masonry or concrete to be reinforced in a satisfactory manner, details to be submitted for approval.
5. We consider the feeder canal embankments near the reservoir as a part of the construction that keeps the water in the reservoir, therefore, the safety of these embankments must receive our consideration. Our Board has not yet concluded its study of these embankments and will ask you to supply us with detailed information regarding them, your plans for further development of the feeders, etc. On one point we have reached a definite

conclusion and you are directed to remove all trees or shrubs which later would become trees, from the embankments of the feeder canals so there will be no likelihood of the embankments becoming breached if the trees on them are blown down by a strong wind.

6. We shall expect you to submit plans and specifications of the changes herein ordered so we may approve them before work is started, all as outlined in Section 3059 of the statutes. If four copies are given to us, we will be able to render a decision on the matter sooner than if a single set, which must be sent around for the individual study of each of the undersigned.

Our Board is aware that the above requirements will be costly, but we believe they are needed to secure the degree of safety which under the statute we are bound to prescribe.

Please acknowledge receipt of this communication to the Chairman of our Board.

Yours truly,

CONN. BOARD OF CIVIL ENGINEERS

By William H. Cadwell  
William H. Cadwell, Chairman

Clarence M. Blair  
Clarence M. Blair, Member

Shepard B. Palmer  
Shepard B. Palmer, Member

Edward W. Bush  
Edward, W. Bush, Member

RECEIVED

MAY 22 1942

STATE WATER CONTROL

Board of Water Commissioners  
City Hall  
New Britain, Conn.

P. O. Box 236, New Haven,  
May 16, 1942

Attn: Mr. M. W. Bannan, Chairman

Re: Shuttle Meadow Dam

Gentlemen:

Your letter of April 1, 1942 addressed to General Sanford H. Wadhams, Chairman of State Board of Supervision of Dams and Reservoirs, was assigned to Mr. S. B. Palmer of Norwich, Secretary of the Board, and the writer, both signers of the original Order, for consideration of your request for a review of the Order of the then Connecticut Board of Civil Engineers dated December 19, 1938 regarding Shuttle Meadow Dam of the City of New Britain. In that order, your Board was directed to make certain changes and repairs before raising the water level above Elevation 372.

That Order made several directions as to repairs, such as an adequate spillway with no provision for flashboards; widening the dam to 20 feet; exploring the present toe of the embankment; and construction of masonry corewall to Elevation 380; and binding the new masonry with the old masonry; removal of trees from embankments, etc.

B-22

These recommendations have not been carried out, but the reservoir has been maintained not higher than Elevation 372, as per the Order of the then Board.

In your letter referred to above, your request now is to be allowed to maintain the reservoir level at Elevation 374, or 2 feet higher than the present allowance.

A conference was arranged for April 16, 1942, and the following were present: General S. H. Wadhams, S. B. Palmer, C. M. Blair, and Mr. W. S. Wise, representing the State and the Board of Supervision of Dams and Reservoirs; and Mr. M. W. Bannan, Chairman, and Mr. J. W. Holden, Engineer, and Mr. Woods, representing the Board of Water Commissioners and the City of New Britain.

It was brought out at this conference that the operation of this Shuttle Meadow Reservoir had undergone certain changes since the hurricane of 1938. At that time, all the watershed of not only Shuttle Meadow itself, of an area of 2.94 square miles, but also the watershed of Whigville, totaling 4 square miles, and the Wolcott watershed of about 2.55 square miles, were also tributary to Shuttle Meadow Reservoir, by means of diverting pipe lines and canals. Since the filtration plant was placed in service a year or so ago, the Whigville watershed was diverted directly to the pumping station below Shuttle Meadow Dam. Plans are also being consummated for diverting the water from the Wolcott watershed around Shuttle Meadow, directly to the pumping station and filtration plant.

As a result of these improvements, we were informed that the Shuttle Meadow Reservoir would only be supplied from its own watershed of about 2.94 square miles. This change in operating conditions suggests another consideration of the Order of the Board.

The trouble at the time of the hurricane was due to a large extent to a heavy storm plus winds of hurricane proportion. The maximum height of water in the reservoir on September 21, 1938, the date of the hurricane, as per data received at that time, was Elevation 376.5. This elevation was substantially  $4\frac{1}{2}$  feet below the top of the concrete coping wall. At that time, the reservoir water was caught by the wind and thrown over the embankment, landing on the downstream slope and causing washouts of the material on the downstream slope.

Our data indicated that this Shuttle Meadow Dam was raised more than once. The dam was constructed in 1891 and was an earth dam with corewall, nearly 600 feet long, and 20 feet wide on top, with upstream slope of 1:2 and downstream slope about 1:1.8. The dam was raised about 4 feet in 1918 and the spillway was changed. The original slopes of the earth embankment were carried up on substantially the same slopes, and the coping wall constructed at the upstream side of the embankment to give a width of the embankment including this wall of about 10 feet. It is our opinion that most of the trouble at the time of the hurricane was due to this comparatively narrow embankment at the top. It is our opinion that under ordinary conditions, the embankment is stable, but the narrowness on top is not good construction, as proved by the hurricane flows.

B-24

It is our studied opinion that the width of the top of embankment should be increased before any change in spillway level of the lake is authorized. We have prepared a sketch showing a suggested method of revising the cross section of the earth embankment before the spillway level is raised to Elevation 374. This suggested plan consists of

• • • • •



constructing a substantial retaining wall at the downstream side of the embankment so as to give a top width of at least 16 feet overall. The turf on the top of the embankment should be removed and additional fill provided on top of this embankment, and then the top turfed again or covered with a stone pavement. The top of the proposed retaining wall should be at least 12 inches higher than the existing coping wall. This proposed retaining wall should extend the entire length of the embankment.

This method of meeting the requirements of the State Board is much simpler than suggested in the Order dated December 19, 1938.

This change Order is made provided that you submit plans and specifications for these changes so that we may approve them before work is started, and also provided that the Shuttle Meadow Reservoir is supplied only from its natural watershed of 2.94 square miles.

A sketch is enclosed to illustrate the proposed treatment of the embankment.

Very truly yours,

*C. M. Blain*  
Member, State Board of Supervision of Dams

OMB:GRB

Approved: \_\_\_\_\_  
Member, State Board of Supervision of Dams

APR 16 1945



HON. GEORGE A. QUIGLEY  
MAYOR

OFFICE OF

STATE WATER COMMISSION  
BOARD OF WATER COMMISSIONERS

CITY HALL, WEST MAIN STREET  
NEW BRITAIN, CONNECTICUT

CHAIRMAN

MAURICE H. PEASE  
JOHN L. HASSON  
PHILIP W. ENGSTRO

NORMAN T. MARSH.  
CHIEF CLERK

JOSEPH W. HOLDEN  
CHIEF ENGINEER

THOMAS F. LUDDY  
SUPERINTENDENT

89-1-1

April 12, 1945

Gen. Sanford H. Wadhams,  
Director, State Water Commission,  
Hartford, Conn.

Dear Sir:

Please accept the appreciation of Mayor Quigley and myself for your promptness and courtesy in visiting Shuttle Meadow reservoir today in regard to the spillway conditions.

After the hurricane of September 21, 1938, and in accordance with the direction of the then State Board of Engineers, the spillway level of the Shuttle Meadow reservoir was lowered from elevation 374 to elevation 372.

Owing to the excessive runoff during the past few months a considerable amount of water was lost and is still being lost over the spillway. At the present elevation of the reservoir one inch of water equals 5.6 million gallons. To save some of this water a twelve-inch flashboard was added raising the spillway elevation one foot to elevation 373 and impounding 67 million gallons.

With the increase in population and accompanying increase in domestic consumption and the unprecedented industrial consumption due to the war effort it became imperative to conserve all water possible. In 1938 the total metered consumption was 241,917,900 cubic feet and in 1944 it was 312,723,300 cubic feet, an increase of 29.3 per cent.

In view of these facts it was felt desirable to add the one-foot flashboard and raise the spillway elevation from 372 to 373.

We request your careful reconsideration of this matter and ask for permission to keep this one-foot flashboard in place as a temporary expedient until next fall.

Mr. Holden informs us that you have the necessary data, including free-board, etc., for the consideration of this problem.

We will prepare an estimate of the cost of the revised recommendation of your Board dated May 16, 1942.

B-26

Very truly yours,

BOARD OF WATER COMMISSIONERS

*William E. Tyler.*  
William E. Tyler, Chairman

K

RECEIVED

MAY 14 1945

STATE WATER COMMISSION

METCALF & EDDY  
Engineers

Statler Building  
Boston 16, Mass.

COPY

May 8, 1945

Board of Water Commissioners  
City Hall  
New Britain, Conn.

Gentlemen:

We are writing to confirm our understanding of the outcome of the discussions last Friday in New Britain, with Mr. V. B. Clarke, member of the State Board of Supervision of Dams, Mayor Quigley, Chairman Tyler and the writer, and later with Mr. Holden, regarding the Shuttle Meadow Dam. Question had been raised by the State Engineers concerning the prudence of using additional flashboards to increase the storage capacity in the reservoir.

Present Conditions. The original dam was raised 4 ft. about 1910, the plans having been approved by the State Authority on September 15, 1909, and the finished work on June 30, 1911. The freeboard between the top of the flashboards on the raised spillway and the earthen top of the dam was about 7.5 feet.; the concrete wall at the top of the upstream slope is 6 inches higher, but does not extend the full length of the dam. Recently about 1 ft. of flashboards has been added, reducing the freeboard to 6.5 ft.

The reservoir is now full and was flowing a few inches over the top of the extra flashboards. An examination of the downstream toe of the dam revealed no evidence of seepage, and indicated that the earthwork of the dam is tight and safe.

B-27

The drainage area above the dam is 1.17 sq. mi.; runoff from additional area is brought in by the west canal, 1.07 sq.mi., and by the east canal 0.7 sq.mi. Facilities, somewhat limited, are provided for diverting the flow of the canals into the stream below the dam.

The waters of the Wolcott Reservoir are brought into Shuttle Meadow Reservoir through twin pipes having a capacity of about 12 million gallons per day; these can be readily shut off.

The spillway at the dam has an effective total width of 18 ft. between the steel stanchions which support the flashboards. This provides overflow capacity of 670 cu.ft. per sec. with a depth of 5 ft. (water surface 2 ft. below the top of the wall). The capacity is believed to be adequate for maximum flood runoff from the watershed of Shuttle Meadow alone, if the canals are otherwise provided for.

During the hurricane of 1938, waves raised by the Southwesterly wind sweeping over the length of the reservoir, about a mile, broke against the wall at the top of the dam, the resulting heavy spray being swept over onto the top and back of the dam and washing away substantial areas of the sods and surface material. The top width of the dam is only 8 ft., which favored damage from this cause.

Proposed Safeguards. It was agreed that the flashboards could be left at their present elevation provided the waste gates on both canals be opened and the Wolcott pipe lines closed. This is to be continued as long as there is any overflow at the Shuttle Meadow dam spillway.

It was urged that existing overflow facilities on the two canals be reviewed and that if necessary auxiliary spillways be

May 8, 1945

constructed to accommodate the maximum expected rates of flood flow independently of the main spillway at Shuttle Meadow Dam. It is understood that Mr. Holden will have measurements made of dimensions and elevations of existing spillways on the canals, to establish the need for and the extent of further overflow capacity.

It was recommended that steps be taken to provided against destructive wave action at the dam, the suggestion being made that this could be advantageously accomplished by constructing a fill of large irregular stone fragments outside the present wall, increasing the width of the crest of the dam to not over 20 ft. and thence sloping down over the present rip-rap at an angle of about 45 degrees. This would not only correct the existing deficiency in top width but would present a steep and irregular slope against which waves would be thoroughly broken, minimizing the density and effect of wind-blown water reaching the top and back slope of the earth dam. Rock for this purpose could be readily quarried out of ledge exposed across the road from the westerly end of the dam, involving a minimum of haul.

It is our opinion that when these matters have been attended to there need be no misgivings as to the adequacy of Shuttle Meadow Dam.

Yours very truly,  
METCALF & EDDY

ALS/C.

By  
Arthur L. Shaw

B-29

April 1, 1948

Memorandum

Shuttle Meadow Reservoir  
March 29, 1948

1. I was requested by Mr. Buck last Friday to consider the run-off factor for the Shuttle Meadow reservoir site.

The run-off factor for use in Myers formula as generally given for this part of the country is 30%. For this watershed then the discharge would be:

$$Q = 30\% (1,000) \sqrt{1.15} = 3220 \text{ cfs.}$$

A discharge so computed is in excess because it is based on maximum flows all over the United States.

A similar type of envelope curve based only on data in New England gives a discharge of 1100 cfs. and one based only on data gathered in Connecticut gives 480 cfs. Mr. Blair once recommended 920 cfs. for areas of this size.

It is thought that the top figure is excessive and the lower figure is not safe, because of lack of data on Connecticut watersheds this small. Therefore, the problem should be approached rationally to determine where in this range a logical design discharge is.

Of the 1.15 sq.mi. of drainage area about Shuttle Meadow 0.26 sq.mi. is the reservoir itself. It seems that the run-off characteristics are so different that the two portions of the area should be considered separately.

The drainage area itself of 1.15 - 0.26 = 0.89 square miles must be considered very fast because of its steepness but mostly because of its circular shape which would allow water from any portion of the watershed to flow quickly and concurrently into the reservoir. According to my report on standard hydrographs this size and type of watershed could reasonably flow at 0.35 (2600) = 910 cfs. Because the peculiar attributes of this shaped area to deliver run-off quickly were not considered in this report I recommend that this figure be increased by 20%, making it 1090 cfs.

The report was based on a maximum rainfall rate of 2.27" per hour. Considering this rate on the 0.26 sq. mi. area, we obtain a volume of 1,380,000 cu.ft. which stated as a flow rate gives 383 cfs.

The total flow rate at the dam then is 1090 + 380 = 1470 cfs. This can be reduced to a run-off factor to use in Myers formula above. This factor would be 14% and may be subject to a factor of safety to be applied by Mr. Buck because, in case of failure, conditions below the dam would make damages great.

This flow is quite large but it is not the only factor to be considered in the spillway design. The storage in the reservoir will affect the necessary provisions for discharge immensely. For example, the total six-hour storm considered as producing such a flood is 6.47 inches. Since the reservoir is .26 ÷ 1.15 = 23% of the total drainage area, a rise of  $\frac{100}{23} (6.47) = 28.1$  inches in the reservoir could store the whole storm with no run-off. The spillway capacity necessary is therefore the capacity that is necessary to discharge sufficient water to reduce 28.1 inches to the surcharge required for safety. The determination of the correct size would be by trial and error computation and could be accomplished if the

limiting lengths and coefficient of the spillway were known.

The contributing areas are also considered "fast" and from the data in my report the following maximum discharges are estimated.

	<u>Drainage Area</u>	<u>Discharge</u>
Shuttle Meadow Reservoir	1.15	1470
Panther Swamp	.70	780
North Canal Area	1.02	1040

All these factors are subject to a factor of safety because of the bad condition for failure which exist.

If Mr. Buck has the limiting design factor on the spillway I could make a stab at a calculation of an outflow hydrograph to determine spillway size required.

Respectfully submitted

*John J. Curry*

J. J. Curry, Sen. San. Engineer

cc for Mr. Buck



UNITED STATES  
DEPARTMENT OF THE INTERIOR  
GEOLOGICAL SURVEY

Water Resources Division  
SV Branch

May 31, 1965

203 Federal Bldg.  
P. O. Box 715  
Hartford 1, Conn.

Mr. George Wood  
Chief Engineer  
Board of Water Commissioners  
Municipal Building  
New Britain, Connecticut

Dear George:

In accordance with our recent discussion on the subject, I have made application of our Connecticut flood-flow formula to Shuttle Meadow Reservoir feeders, obtaining the following peak flows for a flood of 100-year frequency:

Basic Formula:

$$Q_{100} = 3.7 (0.85 \times A \times S)$$

For East Canal

at sill at outlet:  
(0.70 sq. mi.)

$$\begin{aligned} Q_{100} &= 3.7 (0.85 \times 0.70 \times 108) \\ &= 240 \text{ sec.-ft.} \\ &= 240 \text{ sec.-ft. per sq. mi.} \end{aligned}$$

For West Canal

at sill at outlet:  
(1.19 sq. mi.)

$$\begin{aligned} Q_{100} &= 3.7 (0.85 \times 1.19 \times 134) \\ &= 800 \text{ sec.-ft.} \\ &= 420 \text{ sec.-ft. per sq. mi.} \end{aligned}$$

Minor Drainage:

(1.05 sq. miles)

$$\begin{aligned} 1.05 \text{ sq. mi.} \times \frac{240 \text{ cfs} + 420 \text{ cfs}}{2} \\ = 400 \text{ sec.-ft.} \end{aligned}$$

Peak Flow Rate  
into Reservoir  
from Total Tribu-  
tary area of 2.94  
sq. miles

$$\begin{aligned} 240 + 800 + 400 &= 1,440 \text{ sec.-ft.} \\ &= 390 \text{ sec.-ft. per sq. mi.} \end{aligned}$$



Mr. George Wood

-2-

May 31, 1955.

Values of "S" for main water courses were based on profiles of Canals as furnished by you and profiles of natural streams as taken from Geological Survey topographic quadrangle (New Britain sheet). Drainage area figures, "A" in the formula, were used as furnished by you.

For your information, the 100-year flood for Burlington Brook (4.1 sq. miles), computed by this formula, is 1,360 sec.-ft., or 333 sec.-ft. per sq. mile. The value of "S" for Burlington Brook is 105 ft. per mile. For Leadmine Brook (24.0 sq. miles), the 100-year flood by this formula is 220 sec.-ft. per sq. mile. The "S" factor for Leadmine Brook is 72 ft. per mile.

Very truly Yours,

B. L. Figwood,  
District Engineer.

BLB:ors

BENJAMIN H. PALMER  
SHEPARD S. PALMER

**CHANDLER & PALMER**  
CIVIL ENGINEERS  
114-116 THAYER BUILDING  
TELEPHONE TURNER 7-8640

MEMBERS AMERICAN AND CONNECTICUT SOCIETIES  
OF CIVIL ENGINEERS

DAMS  
WATER SUPPLY  
SEWERAGE  
APPRAISALS  
REPORTS  
SURVEYS

NORWICH, CONN.

August 12, 1963

STATE WATER RESOUR  
COMMISSION  
RECEIVED  
AUG 13 1963

ANSWERED.....  
REFERRED.....  
FILED.....

Mr. George W. Wood  
Chief Engineer of Water Department  
City of New Britain  
City Hall  
New Britain, Connecticut

Dear Sir:

This morning I visited the Shuttle Meadow Reservoir in company with your Mr. Naples of the Water Department. This is one of the main sources of supply for the City of New Britain and consists of an earth-filled dam several hundred feet long with a concrete core wall in the center. On the downstream side of the dam, at the base of the slope on the Easterly side of the dam, there is a wet spot perhaps, 75 feet square. In this area the ground is soft and there is some water pushing through the ground and there is evidence of swamp grass growing in this area. On the extreme Westerly side at the base of the dam, there is another similar area but, much smaller, and this is perhaps, 20 feet square. It is my opinion that these are caused by springs in the ground rather than from any leaks actually coming through the dam.

The plan which you gave me indicates that the core wall is down at least sixteen feet deeper than the level of the ground where these wet spots occur. I think it is unlikely that any water is pushing down under this core wall and through the dam. According to the Caretaker and Mr. Naples, this wet condition has existed for a number of years and does not seem to get any worse.

My feeling is that there is no damage to the dam structure and I do not think it is necessary to take any corrective action at this time. If you feel that you want to do anything about it, then

B-34

I think the best thing is to cut a drainage trench from the low part of the spillway flume and fill the trench with coarse gravel carrying it up to the toe of the dam and in this wet area. This would relieve the pressure and get the water downstream without any damage. I don't think it is necessary to do this at the present time, but if your Committee wants to do something, then I think this is the thing to do. The wet spot on the Westerly side is quite small and I think does not need any attention.

Very truly yours,

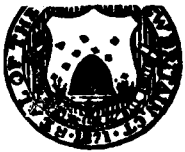
CHANDLER & PALMER

*B. H. Palmer*

B. H. Palmer

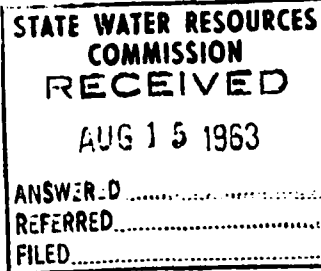
BHP/nir

cc: State Water Resources Commission



CITY OF NEW BRITAIN  
NEW BRITAIN, CONNECTICUT

Water Resources Commission  
State Office Building  
Hartford, Conn.



Attention: Mr. E. A. Dell

Re. Shuttle Meadow Dam -  
New Britain, Conn.

Dear Mr. Dell:

As per your request the Board of Water Commissioners has had the Shuttle Meadow Dam inspected.

You have in your files a report from Chandler and Palmer, Civil Engineers, on this matter.

A further investigation of this dam was conducted by members of the department. On the wet area on the easterly side of the dam there is an observation well. There is no record of this well being installed and apparently has been in existence for a long time. There is an extremely small flow of clear water from this well.

A sounding was made in this well and it was found to be twenty-one feet deep. The temperature of this flowing water was 55°. The temperature of the water in another non-flowing observation well, about one and one-half miles from the first well, was 58°. The same thermometer was used in obtaining the temperature of the water in the reservoir. The thermometer was lowered into the reservoir about 20' and the temperature at this point was 76°.

Will you please advise the Board of Water Commissioners as to any recommendations you may have on this matter.

B-36

Very truly yours,

*George M. Wood*  
George M. Wood

APPENDIX  
SECTION C: DETAIL PHOTOGRAPHS



PHOTO 1 - Hand placed riprap on upstream face below concrete wave wall on crest of dam.

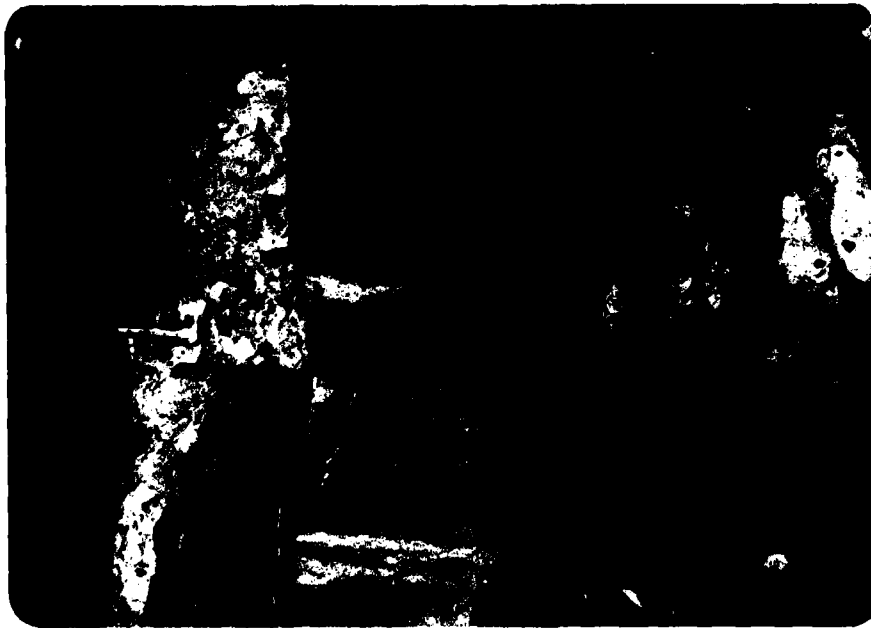


PHOTO 2 - Spalling of upstream face of concrete wave wall. Note fallen chunks of concrete.

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED. DAMS

SHUTTLE MEADOW RESERVOIR  
DAM - WILLOW BROOK  
NEW BRITAIN, CONNECTICUT  
CE# 27 595  
DATE Mar. 79 PAGE C-1



PHOTO 3 - Left diversion inlet. Note cracks in roadway bridge abutment.



PHOTO 4 - Close-up of crack in bridge showing horizontal displacement.

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED. DAMS

SHUTTLE MEADOW RESERVOIR  
DAM - WILLOW BROOK  
NEW BRITAIN, CONNECTICUT  
CE# 27 595  
DATE Mar. 79 PAGE C-2



PHOTO 5 - Spillway approach channel with stoplogs.



PHOTO 6 - View of spillway discharge channel from downstream.  
Note stone lined channel bottom.

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED. DAMS

SHUTTLE MEADOW RESERVOIR  
DAM - WILLOW BROOK  
NEW BRITAIN, CONNECTICUT

CE# 27 595

DATE Mar. 79 PAGE C-3





PHOTO 7 - Right downstream toe of slope in wet area.

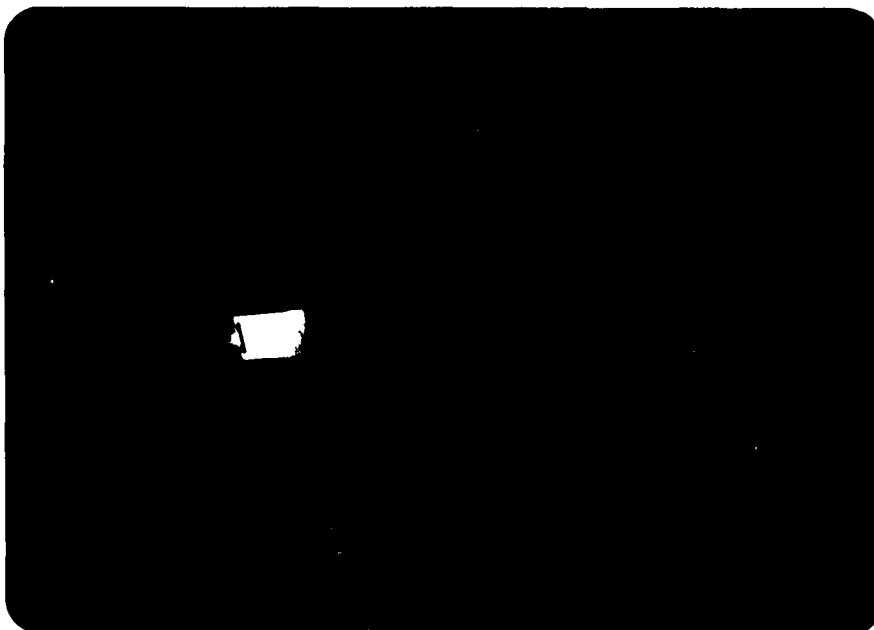


PHOTO 8 - Close-up of wet area at right downstream toe.

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED. DAMS

SHUTTLE MEADOW RESERVOIR  
DAM - WILLOW BROOK

NEW BRITAIN, CONNECTICUT

CE# 27 595

DATE Mar. 79 PAGE C-4



PHOTO 9 - Standpipe at right downstream toe of dam.

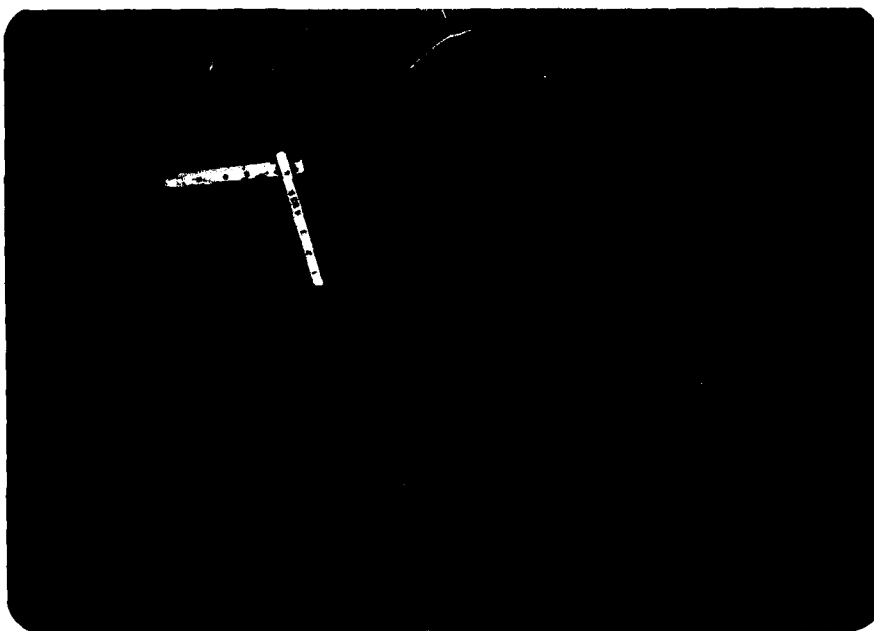


PHOTO 10 - Partially buried stones at left toe of dam. Possible drain outlet.

US ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

CAHN ENGINEERS INC.  
WALLINGFORD, CONN.  
ENGINEER

NATIONAL PROGRAM OF  
INSPECTION OF  
NON-FED. DAMS

SHUTTLE MEADOW RESERVOIR  
DAM - WILLOW BROOK

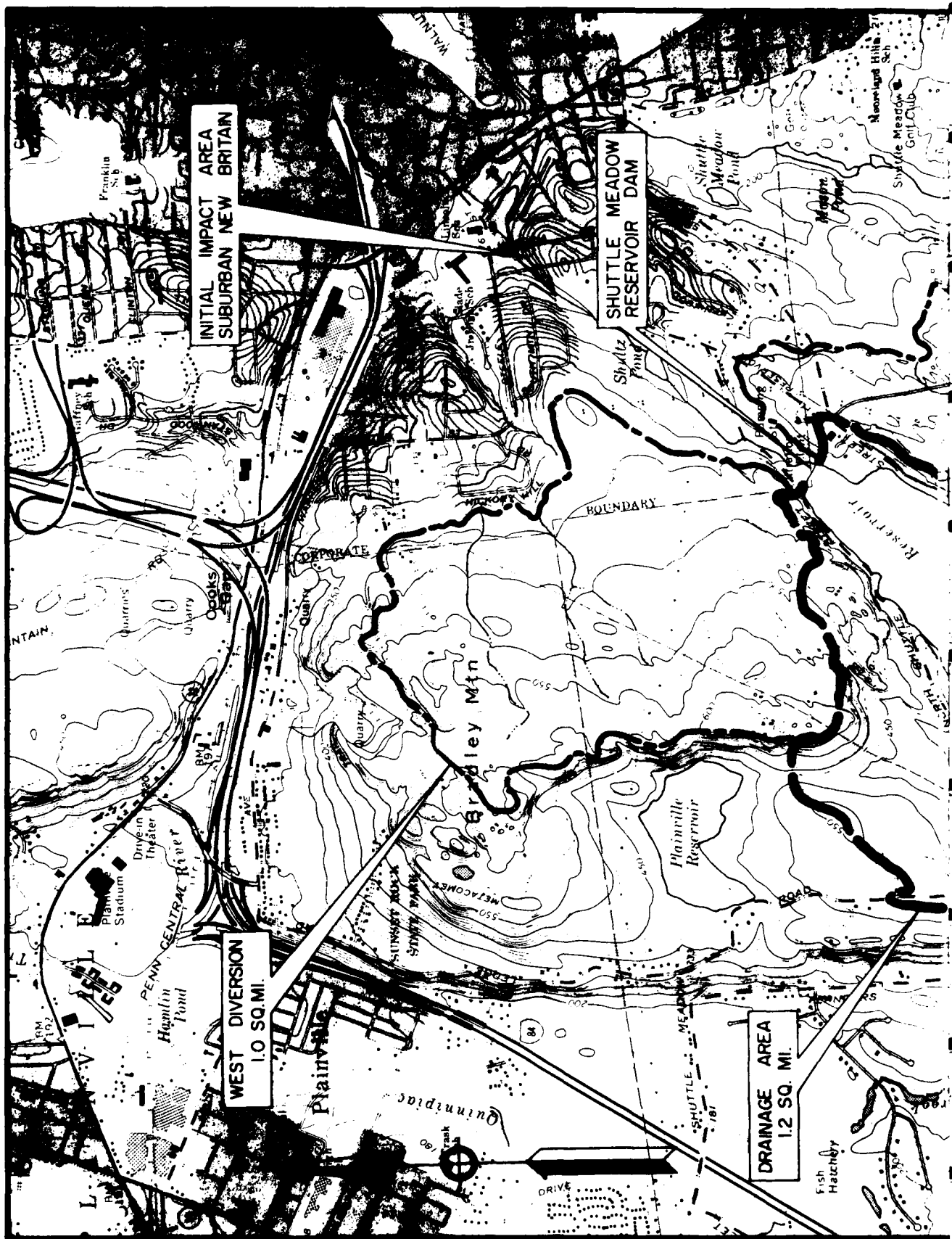
NEW BRITAIN, CONNECTIUCT

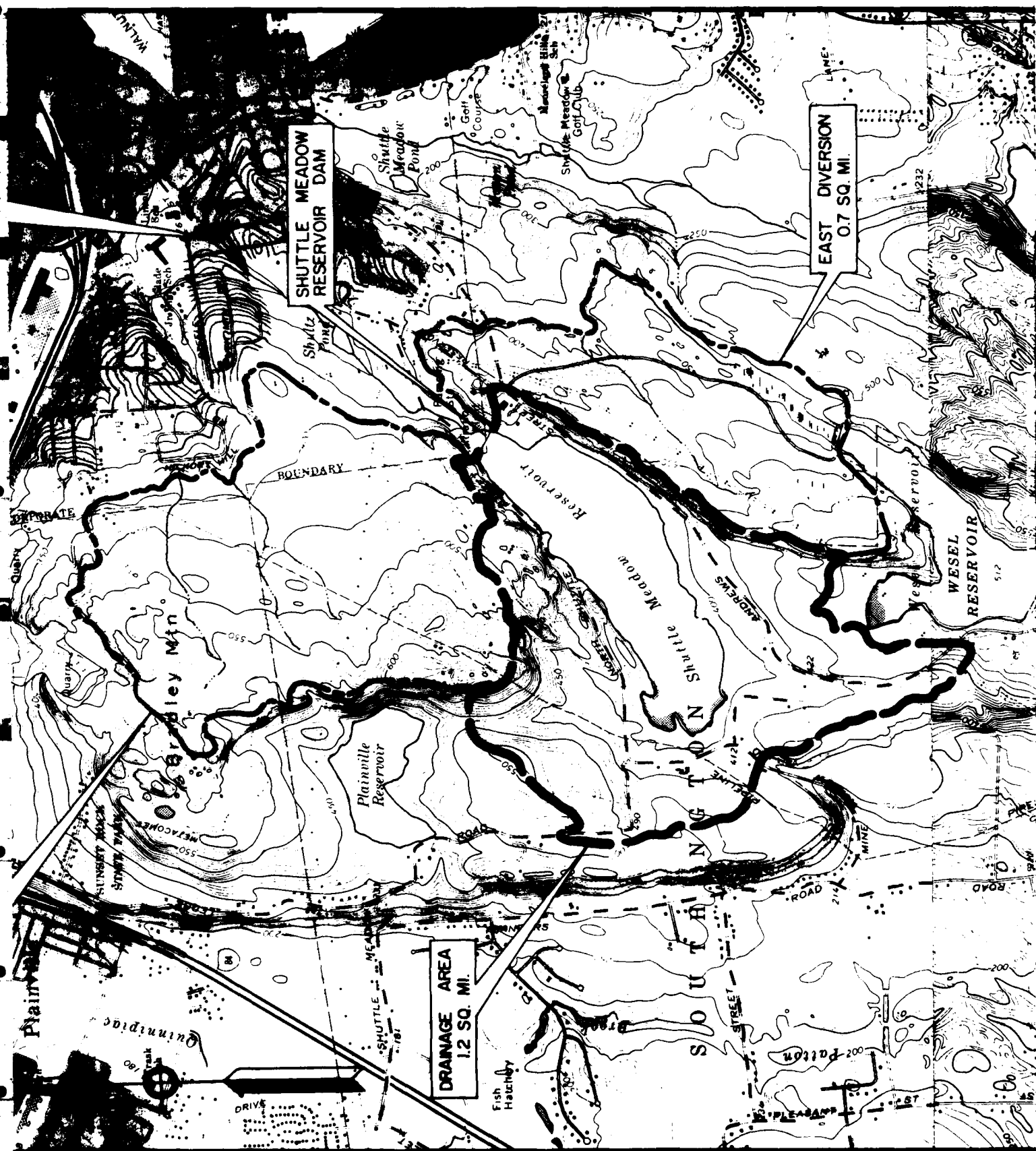
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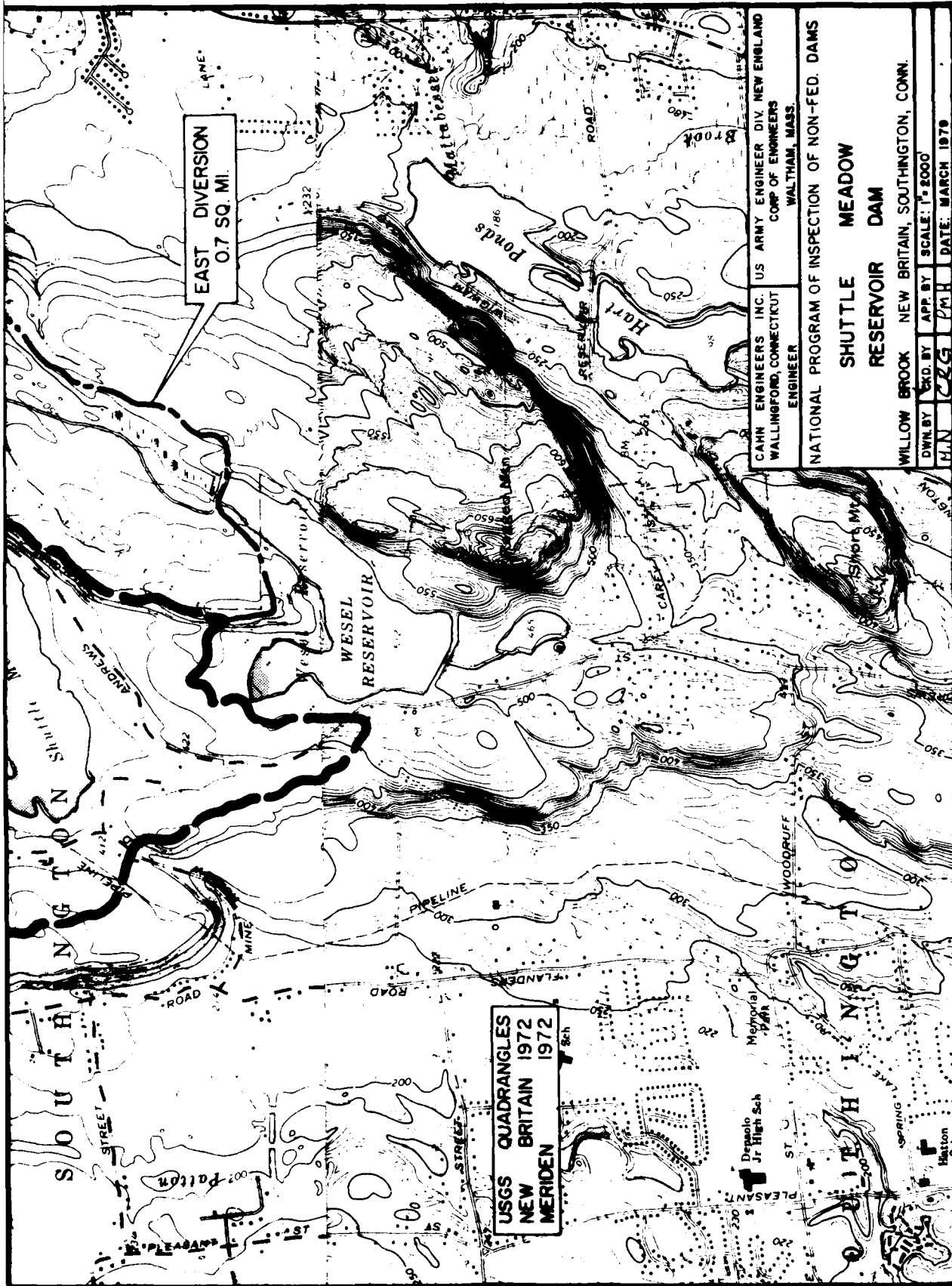
DATE Mar. 79 PAGE C-5

APPENDIX

SECTION D: HYDRAULIC/HYDROLOGIC COMPUTATIONS







**PRELIMINARY GUIDANCE  
FOR ESTIMATING  
MAXIMUM PROBABLE DISCHARGES  
IN  
PHASE I DAM SAFETY  
INVESTIGATIONS**

**New England Division  
Corps of Engineers**

**March 1978**

MAXIMUM PROBABLE FLOOD INFLOWS  
NED RESERVOIRS

<u>Project</u>	<u>Q</u> (cfs)	<u>D.A.</u> (sq. mi.)	<u>MPF</u> cfs/sq. mi.
1. Hall Meadow Brook	26,600	17.2	1,546
2. East Branch	15,500	9.25	1,675
3. Thomaston	158,000	97.2	1,625
4. Northfield Brook	9,000	5.7	1,580
5. Black Rock	35,000	20.4	1,715
6. Hancock Brook	20,700	12.0	1,725
7. Hop Brook	26,400	16.4	1,610
8. Tully	47,000	50.0	940
9. Barre Falls	61,000	55.0	1,109
10. Conant Brook	11,900	7.8	1,525
11. Knightville	160,000	162.0	987
12. Littleville	98,000	52.3	1,870
13. Colebrook River	165,000	118.0	1,400
14. Mad River	30,000	18.2	1,650
15. Sucker Brook	6,500	3.43	1,895
16. Union Village	110,000	126.0	873
17. North Hartland	199,000	220.0	904
18. North Springfield	157,000	158.0	994
19. Ball Mountain	190,000	172.0	1,105
20. Townshend	228,000	106.0(278 total)	820
21. Surry Mountain	63,000	100.0	630
22. Otter Brook	45,000	47.0	957
23. Birch Hill	88,500	175.0	505
24. East Brimfield	73,900	67.5	1,095
25. Westville	38,400	99.5(32 net)	1,200
26. West Thompson	85,000	173.5(74 net)	1,150
27. Hodges Village	35,600	31.1	1,145
28. Buffumville	36,500	26.5	1,377
29. Mansfield Hollow	125,000	159.0	786
30. West Hill	26,000	28.0	928
31. Franklin Falls	210,000	1000.0	210
32. Blackwater	66,500	128.0	520
33. Hopkinton	135,000	426.0	316
34. Everett	68,000	64.0	1,062
35. MacDowell	36,300	44.0	825

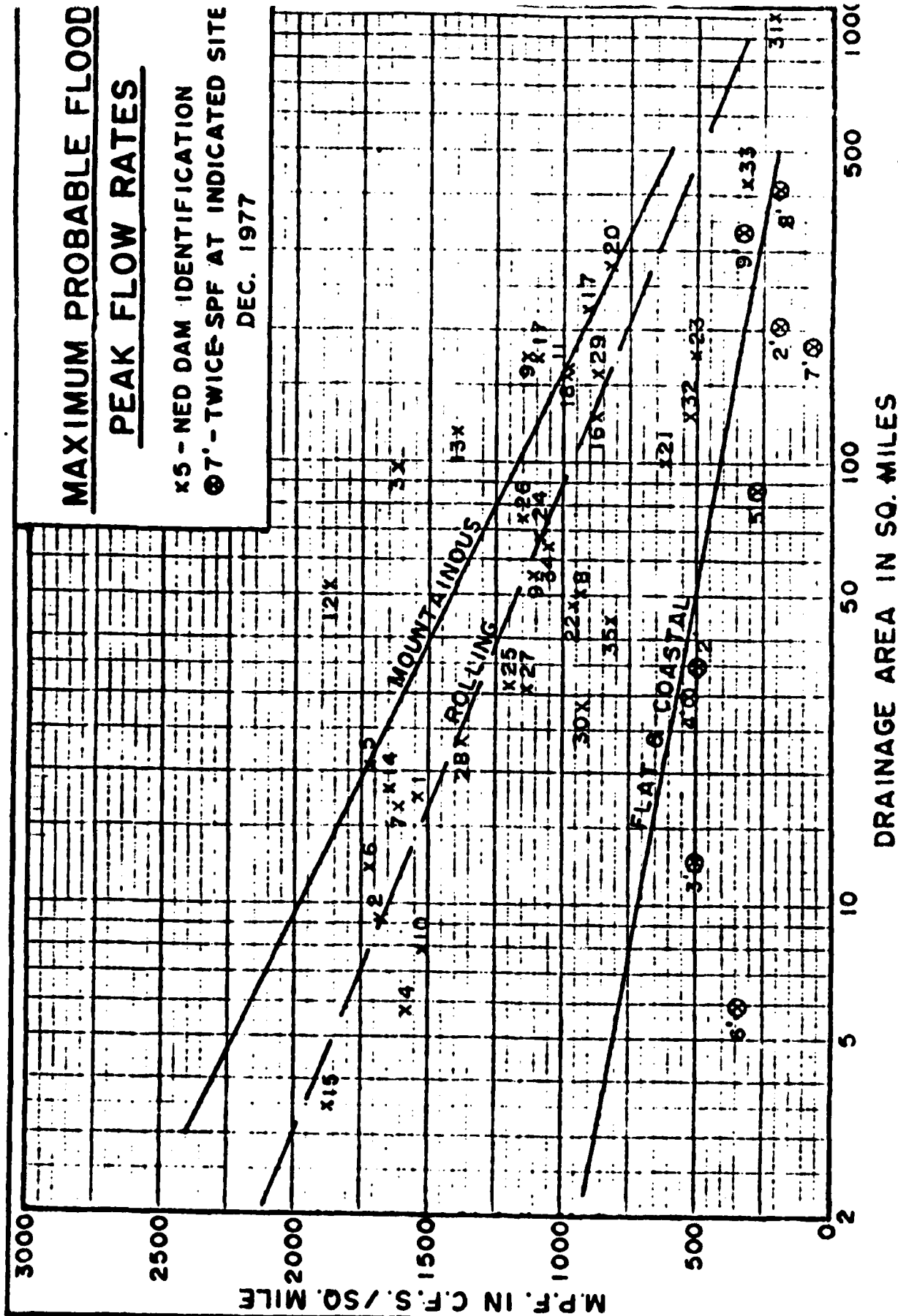


MAXIMUM PROBABLE FLOWS  
BASED ON TWICE THE  
STANDARD PROJECT FLOOD  
(Flat and Coastal Areas)

<u>River</u>	<u>SPF</u> (cfs)	<u>D.A.</u> (sq. mi.)	<u>MPF</u> (cfs/sq. mi.)
1. Pawtuxet River	19,000	200	190
2. Mill River (R.I.)	8,500	34	500
3. Peters River (R.I.)	3,200	13	490
4. Kettle Brook	8,000	30	530
5. Sudbury River.	11,700	86	270
6. Indian Brook (Hopk.)	1,000	5.9	340
7. Charles River.	6,000	184	65
8. Blackstone River.	43,000	416	200
9. Quinebaug River	55,000	331	330

# **MAXIMUM PROBABLE FLOOD PEAK FLOW RATES**

X5 - NED DAM IDENTIFICATION  
 7' - TWICE-SPF AT INDICATED SITE  
 DEC. 1977



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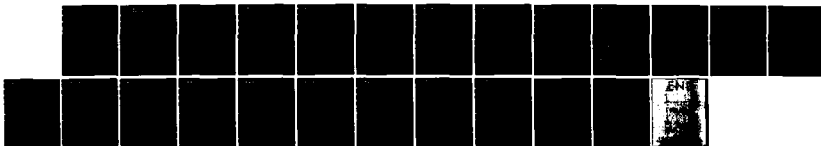
NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS  
SHUTTLE MEADOW RESERV. (U) CORPS OF ENGINEERS WALTHAM  
MA NEW ENGLAND DIV MAR 79

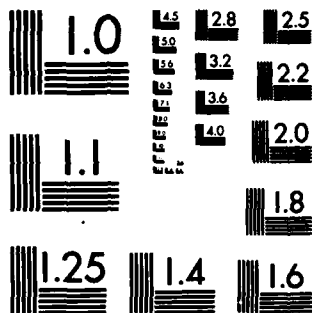
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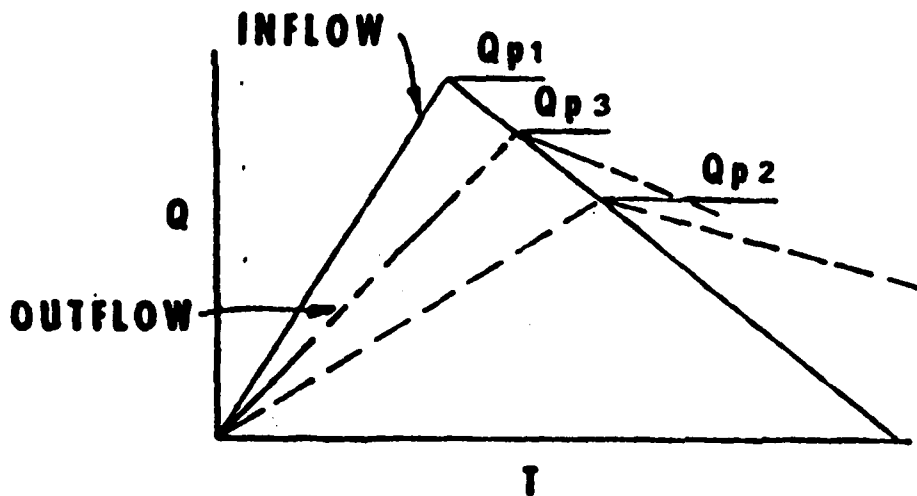
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MICROCOPY RESOLUTION TEST CHART  
NATIONAL BUREAU OF STANDARDS-1963-A

# ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



**STEP 1: Determine Peak Inflow ( $Q_{p1}$ ) from Guide Curves.**

**STEP 2: a. Determine Surcharge Height To Pass " $Q_{p1}$ ".**

**b. Determine Volume of Surcharge ( $STOR_1$ ) In Inches of Runoff.**

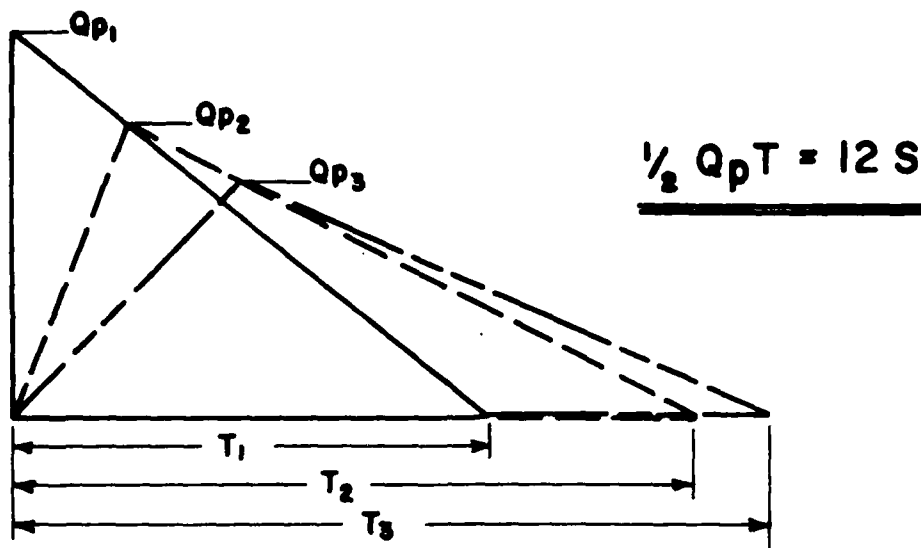
**c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore**

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

**STEP 3: a. Determine Surcharge Height and " $STOR_2$ " To Pass " $Q_{p2}$ "**

**b. Average " $STOR_1$ " and " $STOR_2$ " and Determine Average Surcharge and Resulting Peak Outflow " $Q_{p3}$ ".**

# "RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



**STEP 1:** DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

**STEP 2:** DETERMINE PEAK FAILURE OUTFLOW ( $Q_{p1}$ ).

$$Q_{p1} = \frac{8}{27} W_b \sqrt{g} Y_0^{3/2}$$

$W_b$  = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

$Y_0$  = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

**STEP 3:** USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

**STEP 4:** ESTIMATE REACH OUTFLOW ( $Q_{p2}$ ) USING FOLLOWING ITERATION.

A. APPLY  $Q_{p1}$  TO STAGE RATING, DETERMINE STAGE AND ACCOMPANYING VOLUME ( $V_1$ ) IN REACH IN AC-FT. (NOTE: IF  $V_1$  EXCEEDS  $1/2$  OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL  $Q_{p2}$ .

$$Q_{p2}(\text{TRIAL}) = Q_{p1} \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE  $V_2$  USING  $Q_{p2}(\text{TRIAL})$ .

D. AVERAGE  $V_1$  AND  $V_2$  AND COMPUTE  $Q_{p2}$ .

$$Q_{p2} = Q_{p1} \left(1 - \frac{V_{\text{avg}}}{S}\right)$$

**STEP 5:** FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND  
 Computed By WU Checked By CRG  
 Field Book Ref. \_\_\_\_\_ Other Refs. CE #27-595-KA

Sheet 1 of 19  
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 Revisions \_\_\_\_\_

### HYDROLOGIC/HYDRAULIC INSPECTION

### SHUTTLE MEADOW RESERVOIR DAM, SOUTHWINGTON/NEW BRITAIN, CT.

#### I) PERFORMANCE AT TEST FLOOD CONDITIONS

##### 1) MAXIMUM PROBABLE FLOOD

##### a) WATERSHED CLASSIFIED AS "ROLLING"

##### b) WATERSHED AREA:

SHUTTLE MEADOW RESERVOIR TOTAL WATERSHED INCLUDES ADDITIONAL WATERSHED FEEDING THE RESERVOIR BY CANALS AND PIPELINE. WHILE THE PIPED DIVERSIONS FROM WILCOTT, CAN BE CONTROLLED BY VALVES AT THE INLET END, THE WATERSHED DIVERTED BY CANALS DO NOT HAVE ADEQUATE FACILITIES TO DIVERT THEIR FLOW FROM THE RESERVOIR AND TO THE STREAM BELOW THE DAM. FURTHER, THERE ARE NO FACILITIES TO STOP THE CANALS FLOW INTO THE RESERVOIR (OR, THE RESERVOIR FROM FLOODING THE CANALS). THEREFORE, PARALLEL COMPUTATIONS WILL BE MADE WITH BOTH WATERSHED AREAS:

##### c) WATERSHED INCLUDING D.A.'S DIVERTED BY CANALS INTO THE RESERVOIR:

EAST CANAL (RIGHT):  $(DA)_E = 0.70^{sq\ mi}$

WEST CANAL (LEFT):  $(DA)_W = 1.07^{sq\ mi}$

SHUTTLE MEADOW'S OWN DA:  $(DA)_S = 1.17^{sq\ mi}$

TOTAL  $(DA)_T = \underline{\underline{2.94^{sq\ mi}}}$

ii) WATERSHED EXCLUDING ALL DIVERSIONS:  $(DA)_S = \underline{\underline{1.17^{sq\ mi}}}$

Project NON FEDERAL DAMS INSPECTION

Sheet 2 of 19

Computed By HCL

Checked By CEG

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### SHUTTLE MEADOW RESERVOIR DAM

#### 1,6 - Cont'd) MAXIMUM PROBABLE FLOOD - WATERSHED AREA

MPF ESTIMATED FOR THE SHUTTLE MEADOW RESERVOIR D.A., EXCLUDING ALL DIVERSIONS (1,6,11-P.1), WILL REPRESENT THE CONDITIONS ONLY IF ADEQUATE DIVERSION FACILITIES FOR THE CANAL'S FLOW ARE PROVIDED. THESE FACILITIES SHOULD ALLOW TOTAL DIVERSION OF THE CANALS INFLOW TO THE STREAM D/S FROM THE RESERVOIR.

FLOW FROM PIPED/CONTROLLED DIVERSIONS INTO THE RESERVOIR WILL NOT BE INCLUDED IN THE TEST FLOW AS IT IS ASSUMED THAT THESE DIVERSIONS WILL BE SUSPENDED IF REQUIRED BY TA REGULATION/EMERGENCY OPERATIONAL PROCEDURES OF THE RESERVOIR.

NOTE: WATERSHED DATA (P.1) BY NEW BRITAIN WATER DEPT. AS GIVEN IN E.W.

BUSH REPORT TO CONN. BOARD OF CIVIL ENGINEERS, DATED 10/7/38

AND OTHER MISCELL. CORRESPONDENCE. - U.S.G.S. HARTFORD OFFICE

DATA: (D.A.)<sub>S</sub> = 1.15 sq mi; C.E. FROM U.S.G.S., NEW BRITAIN AND MERIDEN, CT

(1972) QUADRANGLE SHEETS, 1:24000: (D.A.)<sub>E</sub> = 0.59 sq mi (96 FROM WEIR

RESERVOIR); (D.A.)<sub>N</sub> = 1.0 sq mi; (D.A.)<sub>S</sub> = 1.19 sq mi; (D.A.)<sub>T</sub> = 2.78 sq mi

- C) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAX. PROBABLE DISCHARGES" GUIDE CURVE FOR PMF - PEAK FLOW RATES EXTRAPOLATION TO D.A.'S < 2.5 sq mi (AS REQ'D)

$$(PMF)_1 \approx 2000 \text{ cfs/sq mi} \quad (D.A. = 2.94 \text{ sq mi})$$

$$(PMF)_2 \approx 2300 \text{ cfs/sq mi} \quad (D.A. = 1.17 \text{ sq mi})$$

(PMF)<sub>1</sub> IS NOT DIRECTLY APPLICABLE TO THE TOTAL WATERSHED BECAUSE



Project NON-FEDERAL DAMS INSPECTION  
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## SUTTLE MEADOWS RESERVOIR DAM

### 1-Cont'd.) MAXIMUM PROBABLE FLOOD

#### a) PEAK INFLOW

#### i) PMF FOR WATERSHED INCLUDING CANAL DIVERSIONS:

THE PEAK INFLOW OF (2) 60% OF THE CONTRIBUTING WATERSHED ENTERS THE RESERVOIR FROM THE CANALS. THE CANALS, FOR MOST OF THEIR LENGTH, ARE INTERCEPTING CANALS. I.E., THEY RUN PARALLEL TO THE CONTOURS OF THEIR INTERCEPTED HILLSIDE WATERSHED. THEREFORE, THEIR CAPACITY IS LIMITED TO THE TOP OF THE CANALS LOW RILLSIDE BANK. THIS CAPACITY FOR BOTH CANALS, IS CONSIDERABLY LESS THAN THE PMF OF THEIR INTERCEPTED WATERSHEDS, AND VARIES WITH THE W.L. AT THE RESERVOIR AND THE CORRESPONDING BACKWATER AND AVAILABLE FREEBOARD BEFORE THE CANALS ARE, AT SOME POINT OVERTOPPED. (NO SPILLWAY FACILITIES ARE AVAILABLE IN THE CANALS)

ACTUALLY, WHEN THE RESERVOIR IS LOW (I.E. NOT CONTROLLING THE CANALS INFLOW) THE CAPACITY\* OF EACH CANAL FLOWING FULL IS (2) 800 CFS.

FURTHER, NEGLECTING THE EFFECTS OF EXTERNAL STABILITY AND FLOODING WHEN THE W.L. AT THE RESERVOIR REACHES (±) ELEV. 380', THE CANALS ARE OVERTOPPED AND THEREFORE, NO FLOW FROM THEIR INTERCEPTED WATERSHED REACHES THE RESERVOIR. IT IS POSSIBLE AT SOME POINT, A REVERSAL OF THE ENERGY GRADIENT AND CONSEQUENTLY FLOW FROM

\*NOTE: FROM EDWARD W. BUSH REPORT TO CONN. BOARD OF CIVIL ENGINEERS, DATED 10/7/38 ON "SAFETY OF SUTTLE MEADOW DAM" 9.6, THE CANALS CROSS SECTION IS APPROXIMATELY TRAPEZOIDAL (2) 1.5' TO 1.0' SIDE SLOPES, 6' BOTTOM WIDTH; AND, (2) 1"/1000' BOTTOM SLOPE, ASSUMING 11% GRADES AND MAX. DEPTH 4.5' TO 8.5'

Project NON-FEDERAL DAMS INSPECTION

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Computed By JK

Checked By CE

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### SHUTTLE MEADOW RESERVOIR DAM

#### 1, d, i-Cont'd) MAXIMUM PROBABLE FLOOD - PEAK INFLOW - WATERSHED W/CANAL DIVERSION

THE RESERVOIR, OVERFLOWING ALONG THE CANAL WALLS, ABOVE AN ASSUMED (±) ELEV. 380' MSL\*.

DETAILED  $\frac{1}{4}$  ANALYSIS OF THIS CONDITION IS BEYOND THE SCOPE OF A PHASE I INSPECTION. THEREFORE, THE FOLLOWING ASSUMPTIONS WILL BE MADE FOR ESTIMATING THE PMF.

a.) PEAK INFLOW FROM THE CANALS,  $\approx$  (±) 800 CFS.

b.) CANAL INFLOW HYDROGRAPH PROPORTIONAL TO HYDROGRAPH OF D.A. DIRECTLY DRAINING TO RESERVOIR. (ROUGH ASSUMPTION) THEREFORE, R.O. FROM CANALS ASSUMED TO BE EQUIVALENT TO R.O. PRODUCED BY AN EQUIVALENT AREA OF:

$$(DA)_{DIV.} = \frac{1600}{2300} = 0.70 \text{ sq mi.}$$

∴ c.) PEAK INFLOW FROM WATERSHED W/CANAL DIVERSIONS:

$$(PMF)_1 = (1.17 + 0.7) \times 2300 = 1.87 \times 2300 = \underline{\underline{4300 \text{ CFS}}}$$

ii) PMF FOR DIRECT S. MEADOW RES. WATERSHED ( $\frac{1}{2}$  DIVERSIONS)

$$(PMF)_2 = 1.17 \times 2300 = \underline{\underline{2700 \text{ CFS}}}$$

\* MSL ELEVATIONS BASED ON SURVEY FROM NEW BRITAIN WATER (O.B.M. EL. 380.56' MSL (FROM STAP AT LEFT  $\frac{1}{5}$  POST OF BRIDGE OVER SPILLWAY)). THIS ELEV. IS ASSUMED TO CORRELATE WITH TOP OF COPING AT THE DAM, AT EL. 380.92' (MSL) AS SHOWN ON CITY OF NEW BRITAIN, BD. OF WTR. COMM'RS. S. MEADOW DAM MAP DATED OCT. 4, 1938 AND OTHER MISC. CORRESPONDENCE. HOWEVER, B.M. CAN NOT BE THE SAME "IRON STRAP" TO WHICH E.M. BUSN REFERS ON LETTER/REPORT DATED OCT. 7, 1938. (SEE NOTE P.5) D-11

Project NON-FEDERAL DAMS INSPECTION  
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### SHUTTLE MEADOW RESERVOIR DAM

### 2) SPILLWAY DESIGN FLOOD (SDF)

#### a) CLASSIFICATION OF DAM ACCORDING TO HED-ACE RECOMMENDED GUIDELINES.

i) SIZE: STORAGE (MAX)  $\approx 5100 \text{ AC-FT}$  ( $1000 < S < 50000 \text{ AC-FT}$ )  
 HEIGHT  $\approx 38'$  ( $25 < H < 40'$ )

#### STORAGE: ESTIMATED FROM DATA ON PERCY M. BLAKE REPORT TO

THE CONN. ASSOC. OF CIVIL ENGINEERS AND SURVEYORS DATED MAY 31, 1893;  
 OTHER AVAILABLE DATA AND C.E. SURVEY OF 2/26/79:

a.) FROM BLAKE'S REPORT: STORAGE TO ELEV. 60 (CONSTRUCTION DAM) IS  $> 1000 \text{ MG} \approx 3070 \text{ AC-FT}$

b.) CONSTRUCTION ELEV. 60  $\approx$  ELEV. (±) 370' MSL\*

c.) C.E. LAKE AREA MEASURE ON USGS 1:24000' SCALE:

AREA AT ELEV. 373  $\approx 183 \text{ AC.}$  (BLAKE'S AREA @ FLOWLINE 185 AC.)

AREA AT ELEV. 380  $\approx 230 \text{ AC.}$

Avg. AREA  $\approx 206.5$ , SAY 207 AC.

d.) TOP OF DAM (EMBANKMENT) ELEV. 380.4' MSL. (ACTUALLY TO EL. 380' DIRECTION)

e.) MAX. STORAGE  $S_{MAX} \approx 3070 + 207 \times 10 \approx 5100 \text{ AC-FT}$

#### HEIGHT: FROM CITY OF NEW BRITAIN, BOARD OF WATER COMMISSIONERS, SHUTTLE MEADOW DAM MAP DATED 10/9/32

a.) TOP ELEV. OF DAM (EMBANKMENT)  $\approx$  ELEV. 380.4' MSL

b.) ORIGINAL MEADOW LEVEL (SCALED)  $\approx$  ELEV. 342.5' MSL

c.) HEIGHT  $H = 37.9$  SAY, 38' (CORRELATES WELL WITH HEIGHT OF DAM IN 1893 (BLAKE'S  $H = 35' + 4'$  RISE, 19.

\* ACTUALLY ELEV. 370.43' MSL FROM SURVEY BASED ON NEW BRITAIN WATER DEPT. BM. HOWEVER, THERE IS A DISCREPANCY OF (-0.51') BETWEEN THIS AND OTHER KNOWN ELEVATIONS WHICH COULD NOT BE RECONCILED BY MEANS OF THE AVAILABLE SURVEYS AND DATA.

Project NON-FEDERAL DAMS INSPECTION

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Computed By HU

Checked By CPG

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Other Refs. CE #27-595-KA

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## SHUTTLE MEADOW RESERVOIR DAM

### 2. a. Cont'd) CLASSIFICATION

- (i) HAZARD POTENTIAL: THE DAM (AND DIVERSION CANALS) IS LOCATED ( $\pm$ ) 0.9 mi U/S FROM URBANIZED PORTIONS OF NEW BRITAIN, CT. WHERE THE STREAM BECOMES PART OF THE STORM DRAINAGE SYSTEM OF THE CITY SOUTH OF CORBIN AVE. POWER LINES CROSS THE STREAM ( $\pm$ ) 0.5 mi D/F FROM THE DAM.

### (ii) CLASSIFICATION:

SIZE: INTERMEDIATE

HAZARD: HIGH

b) SDF = PMF (SEE 1. d p. 4)

$$(i) (PMF)_1 = 4300 \text{ CFS}$$

$$\frac{1}{2} (PMF)_1 = 2150 \text{ CFS}$$

$$(ii) (PMF)_2 = 2700 \text{ CFS}$$

$$\frac{1}{2} (PMF)_2 = 1350 \text{ CFS}$$

### 3) SURCHARGE AT PEAK INFLOW

a) PEAK INFLOW:  $(Q_p)_1 = 4300 \text{ CFS}$

$$(Q_p')_1 = 2150$$

$$(Q_p)_2 = 2700 \text{ CFS}$$

$$(Q_p')_2 = 1350 \text{ CFS}$$

### b) SPILLWAY (OUTFLOW) RATING CURVE

THE SPILLWAY (WASTEWAY) OF SHUTTLE MEADOW IS ACTUALLY A RECTANGULAR STONE MASONRY CHANNEL ( $\pm$ ) 136' LONG, MOSTLY 20' WIDE

Project NON-FEDERAL DAMS INSPECTION

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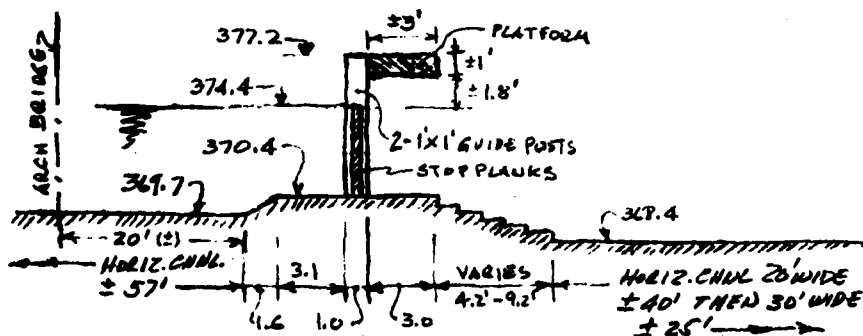
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### SUTTIE MEADOW RESERVOIR DAM

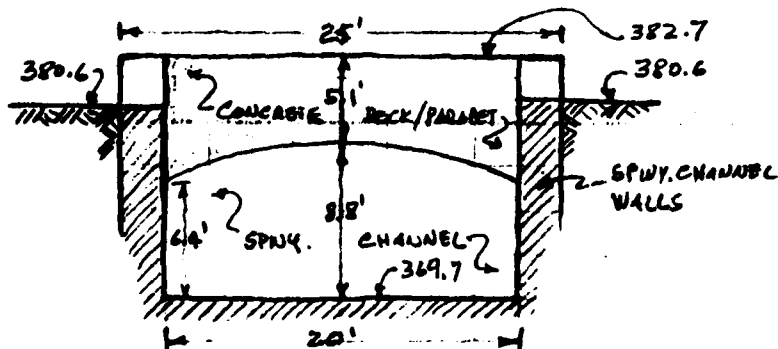
#### 3,6-Cmt'd) OUTFLOW KEYWAY FLOW

WITH A CENTRAL ONE-PIECE SILL 10.0' HIGH FROM THE APPROACH SECTION OF THE CHANNEL. ON THE RIGHT, STOP PLANKS (NOT FISH BOARDS) ARE IN PLACE, RAISING THE ACTUAL CREST ELEVATION OF THE OUTFLOW GATEWAY TO (1) ELEV. 377.2' MSL (12' 4" ABOVE THE PERMANENT SILL ELEV. 370.4' MSL) - (SEE SKETCH) BESIDE 2-STEEL GUIDE POSTS (2" SQ. IN.) FOR THE SUPPORT OF THE PLANKS WHICH EXTEND TO (2) ELEV. 377.2' MSL THERE IS A PLATFORM, 3' WIDE X 1' THICK, WITH TOP ELEV. 377.2' MSL (TO SERVE THE STOP PLANKS) AND AN ARCH BRIDGE (8.8' CLEARANCE AT THE CROWN X 6.4' AT THE SIDES, 20' SPAN), OBSTRUCTING THE CHANNEL.



X-SECTION @ SPWY. SILL

(SPILLWAY CHANNEL 20' WIDE, TOP OF WALLS EL. 380.6' MSL)



DETAIL OF ARCH BRIDGE (13.5' WIDE)

ASSUME TOP OF THE STOP PLANKS AS CREST OF THE "HILLWAY"  
ASSUME  $L = 18'$  (NO FRICTION CORRECTION)  
ASSUME PLATFORM OF OBSTRUCTION REMOVED.

ASSUME WEIR  $C = 3.3$   
ASSUME LOSS/FLOW THRU THE BRIDGE GIVEN BY

$$Q = CA \sqrt{2g(H)} \quad (C=0.7)$$

$$(A \approx 160 \text{ SQ. FT. FULL})$$

ASSUME NO SPILLWAY SUBMERGENCE AND NEGLECT FRICTION AT THE CHANNEL.

Project NON-FEDERAL DAMS INSPECTION

Sheet 8 of 19

Prepared By HLL

Checked By CEG

Date 3/5/79

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### SHUTTLE MEADOW RESERVOIR DAM

### 3,6 - Cont'd) OUTFLOW RATING CURVE

THEREFORE, THE FLOW/DISCH. HEAD OVER THE STOP PLANKS IS APPROX.:

$$Q = 59 H_*^{3/2} \therefore H_* = \left( \frac{Q}{59} \right)^{0.67}$$

THE FLOW/DISCH. HEAD THROUGH THE BRIDGE (ASSUMED FLOWING FULL) IS APPROX.:

$$Q = 900 (\Delta H)^{0.5} \therefore \Delta H = \left( \frac{Q}{900} \right)^2$$

THEREFORE, THE RATING CURVE OF THE SPILLWAY CHANNEL CAN BE ROUGHLY APPROX. BY:

$$H = \left( \frac{Q}{59} \right)^{0.67} + \left( \frac{Q}{900} \right)^2 \quad H \leq 5.6' \text{ (ELEV. } \pm 380' \text{ MSL)}$$

USING THE CREST (TOP OF STOP PLANKS) ELEVATION AS DATUM (ELEV. 374.9' MSL)

(i) EXTENSION OF RATING CURVE FOR SURCHARGE HEADS ABOVE TOP OF DAM.

ACTUALLY, THE TOP OF THE EARTH EMBANKMENT IS AT ELEV 380.4'. TO SIMPLIFY, HOWEVER, ASSUME TOP OF THE EMBANKMENT AT ELEV. 380'. TO MATCH THE ASSUMED OVERTOPPING ELEVATION OF THE DIVERSION CANALS BANKS. ASSUME THIS EMBANKMENT TO BE ( $\pm$ ) 100' LONG TOTAL LENGTH FROM BOTH SIDES OF THE DAM'S COPING WALL TO THE SPILLWAY CHANNEL AT THE RIGHT AND THE ROADWAY AT THE LEFT, AND ( $\pm$ ) 100' ADJ. TO THE DIVERSION CHANNELS. THE COPING WALL (ELEV. 380.9') IS ( $\pm$ ) 460' LONG. THE DIVERSION CANAL LOW HILLSIDE BANKS ARE ASSUMED TO SLOPE AT 0.1% (SAME AS THE CANALS BOTTOM SLOPE).

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SHUTTLE MEADOW KESELYOIK DAM

3,6 - Cont'd) OUTFLOW RATING CURVE

ASSUME  $C=3.0$  FOR THE EARTH EMBANKMENT, COPING WALL  
 AND DIVERSION CANAL BANKS

$C=2.5$  FOR BRIDGE DECK OVERTOP (25' LONG, EL. 382.7' MSL)

ASSUME ALSO, EQUIVALENT LENGTH FOR THE SLOPING CANAL BANKS  
 (TOTAL FOR BOTH CANALS)

$$L'_{R,L} \approx 2 \times \frac{2}{3} (1000) (H-5.6) = 1300 (H-5.6)$$

∴ THE OUTFLOW RATING CURVE, INCLUDING OUTFLOW AT DIVERSION  
 CANALS, MAY BE APPROXIMATED BY:

$$Q_1 \approx Q_s + 600(H-5.6)^{3/2} + 1400(H-6.5)^{3/2} + 1300(H-5.6)^{5/2} + 63(H-8.3)^{3/2}$$

WHERE  $Q_s$  IS THE SPILLWAY CHANNEL DISCHARGE

THE OUTFLOW RATING CURVE ( $Q_1$ ) IS PLOTTED ON P. 10

SIMILARLY, THE OUTFLOW RATING CURVE, W/O DIVERSION CANALS,  
 ASSUMING DAM/CANAL CLOSURES AT ELEV. 380.4' MSL (TOP OF EMBANKMENT)  
 WITH ADDITIONAL HORIZONTAL WEIR LENGTH OF 33' ( $C=3.0$ ); LEFT BANK  
 VERTICAL TO EL. 390 AND 6" : 1" RISE ( $C=2.5$ ) BEYOND; RIGHT BANK  
 ( $C=2.7$ ) AT 6" : 1" RISE, CAN BE APPROXIMATED BY:

$$Q_2 \approx Q_s + 700(H-6)^{3/2} + 1400(H-6.5)^{3/2} + 11(H-6)^{5/2} + 63(H-8.3)^{3/2} + 10(H-15.6)^{5/2}$$

THE OUTFLOW RATING CURVE ( $Q_2$ ) IS ALSO PLOTTED ON P. 10

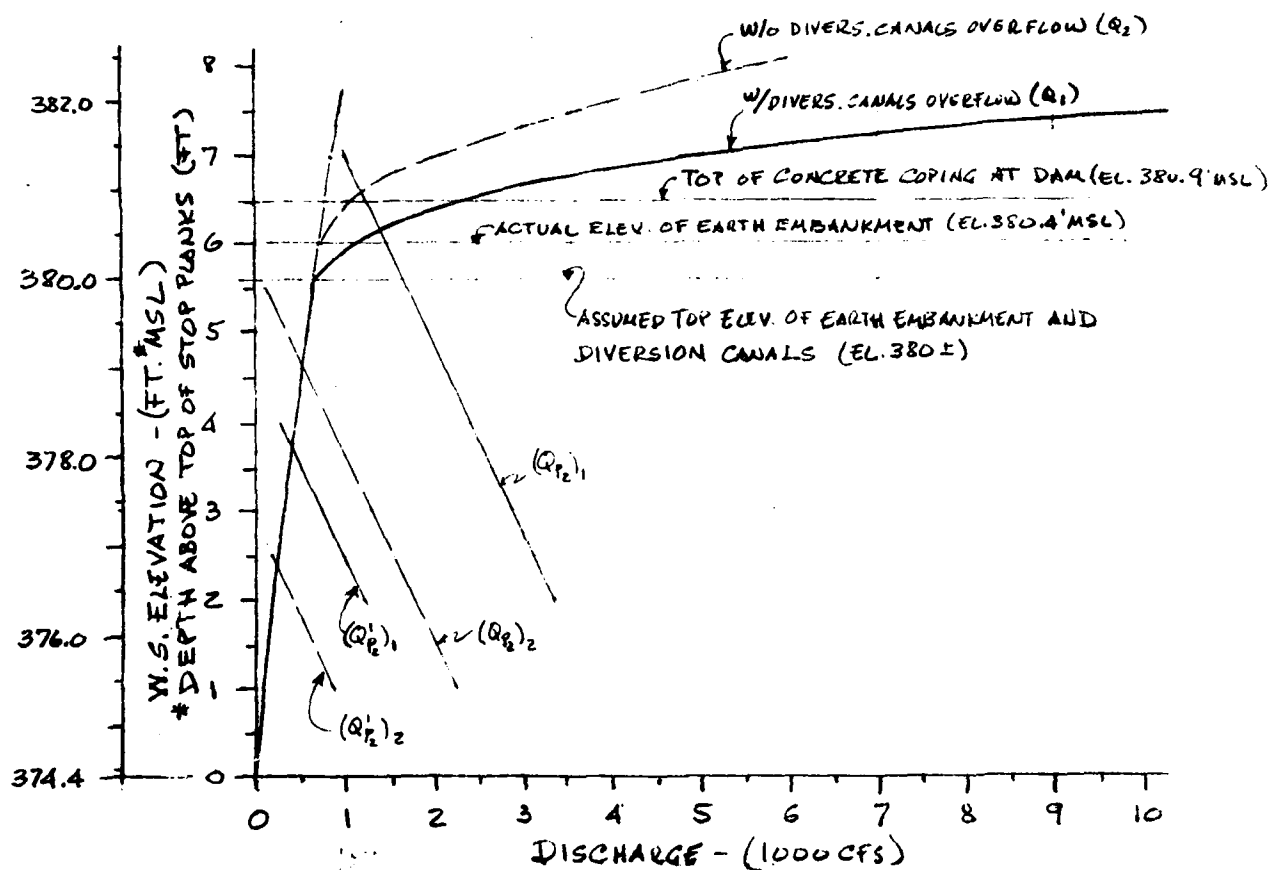
\*NOTE: CANAL CLOSURES / ADJACENT TERRAIN IS ASSUMED TO BE MADE AT TOP EL. OF EMBANK-  
 D-15 MENT.

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### SHUTTLE MEADOW RESERVOIR DAM

### 3-Cont'd) OUTFLOW RATING CURVE



\*NOTES: FOR SOURCE OF MSL ELEV. SEE NOTES ON PP. 4 & 5

DEPTH IS REFERRED TO TOP OF THE STOP PLANKS WHICH IS CONSIDERED THE PRESENT SPILLWAY CREST ELEV. FOR THE DAM. IF THE STOP PLANKS WERE REMOVED THE CREST ELEV. WOULD BE ELEV. 370.4' MSL (4' LOWER) AS THIS IS THE ELEVATION OF THE SPILLWAY CHANNEL SILL. HYDRAULIC CHARACTERISTICS IN THIS CASE WILL BE, FOR RELATIVELY LARGE DEPTHS, MORE THOSE OF A CHANNEL W/O OBSTRUCTIONS/CONSTRICTIONS THAN THOSE GIVEN BY A WEIR FLOW FORMULA. ITS CAPACITY IS EXPECTED HOWEVER TO REMAIN ONLY A SMALL FRACTION OF THE PEAK INFLOW. D-16



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### SHUTTLE MEADOW RESERVOIR DAM

#### 3-Cont'd) SURCHARGE AT PEAK INFLOW

#### c) SPILLWAY CAPACITY TO TOP OF DAM

##### i) EXISTING CONDITIONS (w/CANAL DIVERSIONS):

$$(H)_1 \approx 5.6' \therefore (Q_s)_1 \approx 670 \text{ CFS } (\pm) 16\% \text{ OF } (Q_p)_1 ; (\pm) 31\% \text{ OF } (Q'_p)_1$$

NOTE: SPILLWAY CAPACITY IN THIS CASE IS TAKEN TO ASSUMED TOP OF EMBANKMENT ELEV. 380' MSL, IN ACCORDANCE TO THE ASSUMPTIONS MADE PREVIOUSLY.

SPILLWAY CAPACITY TO ACTUAL TOP OF EMBANKMENT, ELEV. 380.4' MSL, IS  $(\pm) 730 \text{ CFS}$ , AS IN (ii) BELOW. HOWEVER, AT THIS ELEVATION, OVERTOPPING OF THE DIVERSION CANALS, SPECIALLY THE WEST CANAL, HAS PROBABLY OCCURRED.

##### ii) CONDITIONS ASSUMING DIVERSION CANALS CLOSED TO RESERVOIR:

$$(H)_2 \approx 6.0' \therefore (Q_s)_2 \approx 730 \text{ CFS } (\pm) 27\% \text{ OF } (Q_p)_2 ; (\pm) 54\% \text{ OF } (Q'_p)_2$$

#### d) SURCHARGE HEIGHT TO PASS $(Q_p)$

$$i) @ (Q_p)_1 = (PMF)_1 \approx 4300 \text{ CFS } (H_1)_1 = 6.9' \text{ (w/CANALS OVERFLOW)}$$

$$@ (Q'_p)_1 = \frac{1}{2} (PMF)_1 \approx 2150 \text{ CFS } (H'_1)_1 = 6.5' \text{ (w/CANALS OVERFLOW)}$$

NOTE: THE RELATIVELY LOW SURCHARGE AND SMALL CHANGE AMONG THEM IS MAINLY DUE TO THE LONG LENGTH OF SPILL ALONG THE DIVERSION CANAL BANKS.  $(\pm 1700' @ (H_1)_1 = 6.9')$

$$ii) @ (Q_p)_2 = (PMF)_2 \approx 2700 \text{ CFS } (H_1)_2 = 7.2' \text{ (w/CANALS OVERFLOW)}$$

$$@ (Q'_p)_2 = \frac{1}{2} (PMF)_2 \approx 1350 \text{ CFS } (H'_1)_2 = 6.7' \text{ (w/CANAL OVERFLOW)}$$

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### SHUTTLE MEADOW RESERVOIR DAM

#### 4) EFFECT OF SURCHARGE STORAGE ON MAX. PROBABLE DISCHARGE (OUTFLOW)

a) RESERVOIR (LAKE) AGE AREA WITHIN EXPECTED SURCHARGE:  $A_s = 207^{AC}$ .  
(SEE P. 5)

b) ASSUME NORMAL POOL LEVEL AT STOP PLANKS TOP ELEV. (EL. 374.4' MS)

c) WATERSHED AREA:

i) W/CANAL DIVERSIONS: EQUIV. D.A. = 1.87<sup>sq mi</sup> (SEE P. 4)

ii) W/O CANAL DIVERSIONS: D.A. = 1.17<sup>sq mi</sup> (DIRECT D.A. TO RESERV.)

d) DISCHARGE ( $Q_p$ ) AT VARIOUS SURCHARGE ELEVATIONS:

i) W/CANAL DIVERSIONS:

$$(H)_1 = 7' \quad (V)_1 = 207 \times 7 = 1450^{ACFT} \therefore (S)_1 = \frac{1450}{1.87 \times 53.3} = 14.5''$$

$$(H)_1 = 4' \quad (V)_1 = 828^{ACFT} \therefore (S)_1 = 8.31''$$

$$(H)_1 = 2' \quad (V)_1 = 414^{ACFT} \therefore (S)_1 = 4.15''$$

$\therefore$  FROM APPROX. STORAGE ROUTING NED-ACE GUIDELINES (19" MAX. PROBABLE R.O. IN NEW ENGLAND)

$$Q_p' = Q_p \left(1 - \frac{S}{19}\right) \text{ AND FOR } Q_p' = \frac{1}{2} \text{ PMF: } Q_p' = Q_p \left(1 - \frac{S}{19}\right)$$

FOR:

$$(H)_1 = 7' \quad (Q_p)_1 = 1010^{CFS}$$

$$(H)_1 = 4' \quad (Q_p)_1 = 2420^{CFS}$$

$$(H)_1 = 2' \quad (Q_p)_1 = 3360^{CFS}$$

$$(Q_p')_1 = 270^{CFS}$$

$$(Q_p')_1 = 1210^{CFS}$$

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### SHUTTLE MEADOW RESERVOIR DAM

4, d-Cont'd) DISCHARGE ( $Q_2$ ) AT VARIOUS SURCHARGE ELEVATIONS:

(i) W/O CANAL DIVERSIONS:

$$(H)_2 = 5.5' \quad (V)_2 = 207 \times 5.5 \approx 1140 \text{ ACFT} \quad \therefore (S)_2 = \frac{1140}{1.17 \times 53.3} = 18.3''$$

$$(H)_2 = 2.5' \quad (V)_2 = 518 \text{ ACFT} \quad \therefore (S)_2 = 8.30''$$

$$(H)_2 = 2' \quad (V)_2 = 414 \text{ ACFT} \quad \therefore (S)_2 = 6.64''$$

$$(H)_2 = 1' \quad (V)_2 = 207 \text{ ACFT} \quad \therefore (S)_2 = 3.32''$$

$\therefore Q_{P2}$  AND  $Q'_{P2}$  FOR RESERV. W/O DIVERSIONS (AS IN 4, d, i p. 12):

FOR:

$$(H)_2 = 5.5' \quad (Q_{P2})_2 = 106 \text{ CFS}$$

$$(H)_2 = 2.5' \quad (Q_{P2})_2 = 1520 \text{ CFS} \quad (Q'_{P2})_2 = 171 \text{ CFS}$$

$$(H)_2 = 2' \quad (Q_{P2})_2 = 1760 \text{ CFS} \quad (Q'_{P2})_2 = 207 \text{ CFS}$$

$$(H)_2 = 1' \quad (Q_{P2})_2 = 2230 \text{ CFS} \quad (Q'_{P2})_2 = 878 \text{ CFS}$$

### e) PEAK OUTFLOW ( $Q_{P3}$ )

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALT. METHOD (SEP. 10)

(i) W/CANAL DIVERSIONS:

$$(Q_{P3})_1 \approx 1400 \text{ CFS} \quad (H_3)_1 \approx 6.2' \quad \text{FOR } (Q_{P1})_1 = (PMF)_1 \approx 4300 \text{ CFS}$$

$$(Q'_{P3})_1 \approx 390 \text{ CFS} \quad (H'_3)_1 \approx 3.7' \quad \text{FOR } (Q'_1)_1 = \frac{1}{2}(PMF)_1 \approx 2150 \text{ CFS}$$

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### SHUTTLE MEADOW RESERVOIR DAM

#### 4. C-CANAL) PEAK OUTFLOW ( $Q_P$ )

##### (i) W/O CANAL DIVERSIONS:

$$(Q_P)_2 \approx 520 \text{ CFS} \quad (H_3)_2 \approx 4.6' \quad \text{FOR } (Q_P)_2 = (PMF)_2 \approx 2700 \text{ CFS}$$

$$(Q'_P)_2 \approx 210 \text{ CFS} \quad (H_3')_2 \approx 2.4' \quad \text{FOR } (Q'_P)_2 = \frac{1}{2}(PMF)_2 \approx 1350 \text{ CFS}$$

#### 4) SPILLWAY CAPACITY RATIO TO OUTFLOW:

##### (i) EXISTING CONDITIONS (W/CANAL DIVERSIONS) - SPILLWAY CAPACITY TO TOP OF EARTH EMBANKMENT / CANALS BANKS (ASSUMED TO BE AT ELEV. 380' MSL):

$$(Q_S)_1 \approx 670 \text{ CFS}$$

∴ SPILLWAY CAPACITY (i) IS (±) 48% THE OUTFLOW @ (PMF)<sub>1</sub> AND (±) 170% THE OUTFLOW @  $\frac{1}{2}(PMF)_1$

##### (ii) W/O CANAL DIVERSIONS: - SPILLWAY CAPACITY TO TOP OF EARTH EMBANKMENT (ASSUMED LOWEST OVERTOPPING PT.) - ELEV. 380.4' MSL:

$$(Q_S)_2 \approx 730 \text{ CFS}$$

∴ SPILLWAY CAPACITY (ii) IS (±) 140% THE OUTFLOW @ (PMF)<sub>2</sub> AND (±) 350% THE OUTFLOW AT  $\frac{1}{2}(PMF)_2$

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### SHUTTLE MEADOW RESERVOIR DAM

#### I-Cont'd) PERFORMANCE AT TEST FLOOD CONDITIONS

#### 5) SUMMARY

##### a) PEAK INFLOW:

i) W/CANAL DIVERSIONS:  $(Q_P)_1 = (PMF)_1 = 4300 \text{ cfs}$   $(Q'_P)_1 = \frac{1}{2} (PMF)_1 = 2150 \text{ cfs}$

ii) W/O CANAL DIVERSIONS:  $(Q_P)_2 = (PMF)_2 = 2700 \text{ cfs}$   $(Q'_P)_2 = \frac{1}{2} (PMF)_2 = 1350 \text{ cfs}$

##### b) PEAK OUTFLOW:

i) W/CANAL DIVERSIONS:  $(Q_P)_1 = 1400 \text{ cfs}$   $(Q'_P)_1 = 390 \text{ cfs}$

ii) W/O CANAL DIVERSIONS:  $(Q_P)_2 = 520 \text{ cfs}$   $(Q'_P)_2 = 210 \text{ cfs}$

##### c) SPILLWAY MAX. CAPACITY

i) EXIST. CONDITIONS (W/CANAL DIVERSIONS):  $(Q_S)_1 = 670 \text{ cfs}$  OR, (1) 48% OF  $(Q_P)_1$   
AND (2) 170% OF  $(Q'_P)_1$

THEREFORE, AT  $(SDF)_1 = (PMF)_1$ , THE DAM/CANALS ARE PROBABLY OVERTOPPED (1) 0.6' (U.S. EL. 380.6' MSL) OR TO AN AVE. SURCHARGE ABOVE THE SPILLWAY STOP PLANKS OF (1) 6.2'.

ii) W/O CANAL DIVERSIONS:  $(Q_S)_2 = 730 \text{ cfs}$  OR, (1) 140% OF  $(Q_P)_2$  AND  
(2) 350% OF  $(Q'_P)_2$

THEREFORE, AT  $(SDF)_2 = (PMF)_2$  THE DAM WILL PROBABLY NOT BE OVERTOPPED. (U.S. EL. 379.0' MSL) OR AN AVE. SURCHARGE ABOVE THE SPILLWAY STOP PLANKS OF (1) 4.6'.

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### SHUTTLE MEADOW RESERVOIR DAM.

### II) DOWNSTREAM FAILURE HAZARD

#### 1) PEAK FLOOD AND STAGE IMMEDIATELY $\frac{1}{2}$ FROM DAM

##### a) BREACH WIDTH:

i) MID-HEIGHT (?) ELEV. 362' MSL  $(380.9 - \frac{38}{2} = 361.9'$  SAY 362' MSL)  
(\*SEE P. 5)

ii) APPROX. MID-HEIGHT LENGTH  $L \approx 410'$  (C.E. MEASURED ON C.E. INSPECTION DAM.)

iii) BREACH WIDTH (SEE NEG. ACE  $\frac{1}{2}$  DAM FAILURE SURVEILLANCE):

$$W = 0.4 \times 410 = 164' \therefore \text{ASSUME } W_b = \underline{160'}$$

#### b) PEAK FAILURE OUTFLOW ( $Q_p$ )

ASSUME SURCHARGE TO ASSUMED TOP OF EMBANKMENT ELEV. 380.4' MSL.

i) HEIGHT AT TIME OF FAILURE:  $Y_o \approx 37.5'$  SAY  $Y_o = 38'$

ii) SPILLWAY / CANAL OVERFLOW DISCHARGE:  $Q_{s,c} \approx 1000 \text{ CFS.}$

iii) BREACH OUTFLOW ( $Q_b$ ):  $Q_b = \frac{P}{27} W_b \sqrt{g} Y_o^{3/2} \approx 65000 \text{ CFS}$

iv) PEAK FAILURE OUTFLOW ( $Q_p$ ):  $Q_p = Q_{s,c} + Q_b \approx \underline{64000 \text{ CFS}}$

\*NOTE: BECAUSE OF THE CANAL DIVERSIONS, IF FAILURE IS ASSUMED TO A  $\frac{1}{2}$  TOP OF EMBANKMENT THE OVERFLOW (SPILLWAY / DIV. CANAL BANKS) COULD BE (?) 1000 CFS

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### SHUTTLE MEADOW RESERVOIR DAM

1-Cont'd) PEAK FLOOD AND STAGE IMMEDIATELY D/S FROM DAM

c) FLOOD WAVE HEIGHT IMMEDIATELY D/S OF DAM:

$$y = 0.44 y_0 = 17'$$

2) ESTIMATE OF D/S DAM FAILURE CONDITIONS AT IMPACT AREA

(SEE NED-ACE GUIDELINES FOR ESTIMATING D/S DAM FAILURE HYDROLOGY)

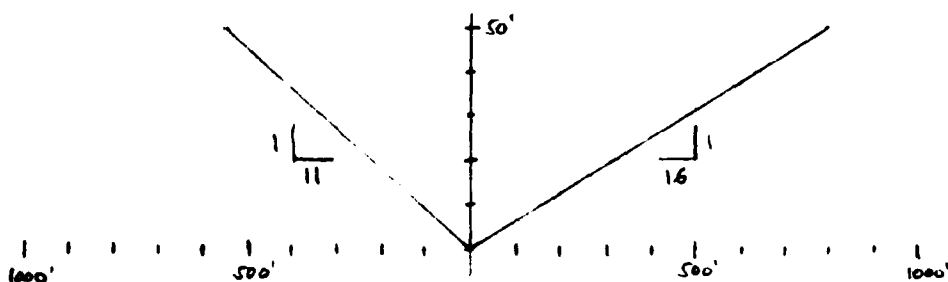
ASSUME RESERVOIR FULL TO TOP OF EMBANKMENT AT TIME OF FAILURE

a) RESERVOIR STORAGE AT TIME OF FAILURE:  $S = 5100$  ACFT (SEE P. 5)

$$S/2 = 2500 \text{ ACFT}$$

b) TYPICAL D/S CROSS SECTION & RATING CURVES:

(FROM USGS, NEW BRITAIN, CT., QUADRANGLE SHEET, PHOTO REV. 1972, SCALE 1:24000)



ASSUME: 1)  $n = 0.050$

2) SLOPE:  $S_0 = 2.56\%$

(DROPS 100' IN (1) 3900')

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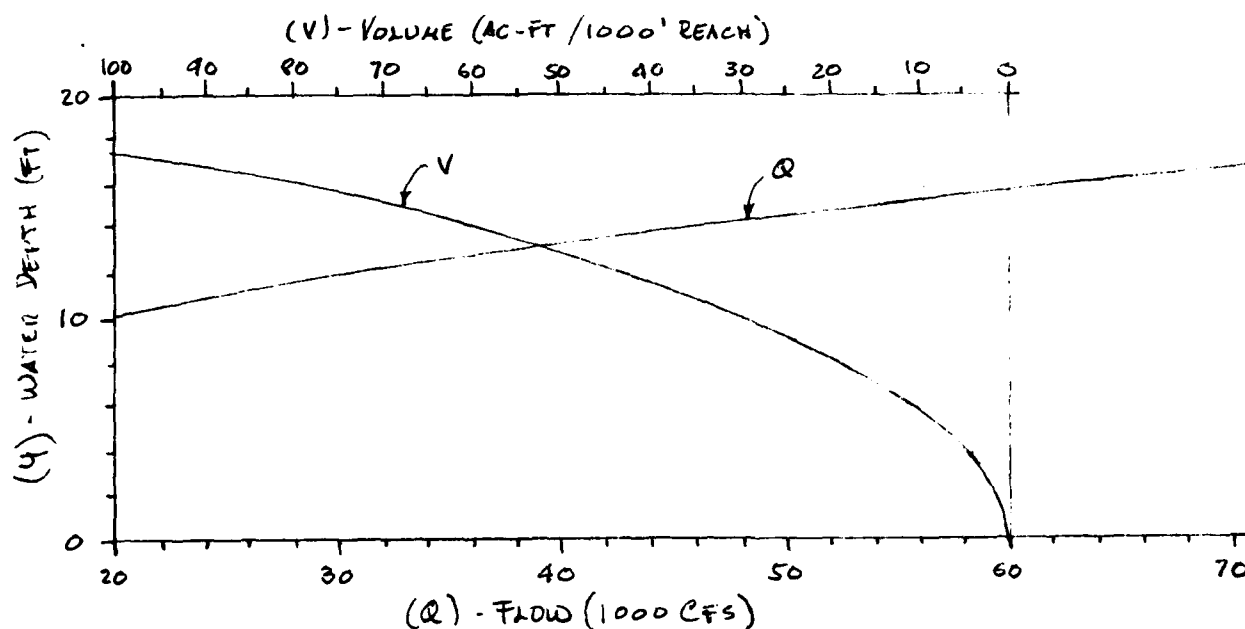
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### SHUTTLE MEADOW RESERVOIR DAM

#### 2- Cont'd) P/S DAM FAILURE CONDITIONS AT IMPACT AREA

#### C) RATING CURVES (P/S CROSS SECTION)



#### d) REACH OUTFLOW ( $Q_p$ )

i) ASSUME REACH LENGTH  $L = 5100'$  (S. MEADOWS TO URBAN AREA OF NEW BRUNSWICK)

ii) @  $Q_p = 64000$  CFS  $\therefore y_1 = 16'$   $\therefore V_1 = 400$  AC-FT  $\leq \frac{L}{5}$  OK

iii)  $Q_p = Q_1 (1 - \frac{V_1}{L}) = 59000$  CFS  $\therefore y_2 = 15.4'$   $V_2 = 370$  AC-FT

iv) ADE VOLUME IN REACH  $V_{ADE} = 385$  AC-FT

v)  $\therefore Q_p = 59200$  CFS  $\therefore y_2 = 15.4'$  (AT IMPACT AREA)



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### SHUTTLE MEADOW RESERVOIR DAM

### II - (Cont'd) DOWNSTREAM FAILURE HAZARD

#### 3) SUMMARY

a) PEAK FAILURE OUTFLOW:  $Q_p \approx 64000 \text{ cfs}$

b) REACH OUTFLOW:  $Q_{p, \text{reach}} = 59200 \text{ cfs}$

c) AVE. WATER DEPTH (STAGE)  $y_{\frac{1}{3}} = 15.4'$

APPENDIX

SECTION E: INFORMATION AS CONTAINED IN THE  
NATIONAL INVENTORY OF DAMS



## INVENTORY OF DAMS IN THE UNITED STATES

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IDENTITY DIVISION NUMBER	162	STATE	CT	COUNTY	003	CORR	006	NAME	SHUTTLE MEADOW RESERVOIR DAM	LATITUDE (NORTH)	41 38.7	LONGITUDE (WEST)	72 49.2	REPORT DATE DAY   MO   YR	26 DEC 73
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New Britain

G-8

POPULAR NAME	NAME OF IMPOUNDMENT
	SHUTTLE MEADOW RESERVOIR

REGION	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01 008	WILLOW BROOK	NEW BRITAIN	1	62200

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEAD (FT.)	IMPOUNDING CAPACITIES (ACRE-FT.)	DIST DOWN FLD R (MI.)	VER/DATE
MECH	1930	S	16	14	640	576	NED

REMARKS
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D/S HAS LENGTH	545	SPILLWAY TYPE	20	VOLUME OF DAM (CV)	POWER CAPACITY INSTALLED (KW)	PROPOSED (KW)	NO.	LENGTH OF ELEVATION	125	WIDTH	125	DEPTH	125	WIDTH	125
----------------	-----	---------------	----	--------------------	-------------------------------	---------------	-----	---------------------	-----	-------	-----	-------	-----	-------	-----

OWNER	ENGINEERING BY	CONSTRUCTION BY
NEW BRITAIN WATER CO		

DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE

INSPECTION BY	INSPECTION DATE DAY   MO   YR	AUTHORITY FOR INSPECTION
DEPT ENV PHOT	24 MAY 73	PA 571 SECT 25-11 ST UP CT

REMARKS
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END

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9-84

DTIC